Vertical Interior Post and Horizontal Rail Design

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By MJF Date 10/7/21
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$H_{post} := 42\ in$ Plans

$F_y := 50\ ksi$ ASTM F1043

$E_s := 29000\ ksi$

$F_u := 58\ ksi$

...Pipe for Interior Posts

$OD_{post} := 2.875\ in$

$t_{post} := 0.160\ in$

Properties for ASTM F1043 IC 1-5/8" Pipe for Rails

Rail OD: $OD_{rail} := 1.66\ in$

Rail Thickness: $t_{rail} := 0.111\ in$

Design Point Live Load $P_{LL} := 200\ lbf$ AASHTO 13.8.2

Design Uniform Live Load $w_{LL} := 50\ plf$ AASHTO 13.8.2

Post spacing: $L_{spc} := 6\ ft$ Plans

Weight of chain link fence: $f_{clf} := 0.48\ psf$

Design wind load from chain link fence: $f_{wind} := 15\ psf$ AASHTO 13.8.2

Load Factors (AASHTO Tbl. 3.4.1-1):

PL Load Factor: $\gamma_{PL} := 1.75$

DC Load Factor: $\gamma_{DL} := 1.25$

WS Load Factor: $\gamma_{WS} := 1.00$

Resistance Factors:

Steel Flexure (AASHTO 6.5.4.2): $\phi_f := 1.00$

Steel Shear (AASHTO 6.5.4.2): $\phi_v := 1.00$

Tension, Yielding in Gross Section: $\phi_y := 0.95$

Bending (AISC F1): $\phi_b := 0.90$

Shear (AISC G1): $\phi_{v\_AISC} := 0.90$

Bearing (AISC DG#1): $\phi_{brg} := 0.60$

Fillet Weld (AISC Tbl. J2.5): $\phi_{fw} := 0.75$

Bolts (AISC J3.6 & J3.7): $\phi_{ab} := 0.75$

Adhesive Anchor Bolts (ACI 17.3.3, Condition B, Category 1): $\phi_{adh} := 0.65$

Steel weight density: $\gamma_{steel} := 490\ pcf$

ASTM F1043 Group IC Electric Resistant Welded 50,000 psi yield steel pipe																
Trade Reference	Decimal O.D. Equivalent		Pipe wall Thickness		Weight		Section Modulus		x	Min. Yield Strength		=	Max Bending Moment	Calculated Load (lbs)		
														10' Free Supported	Cantilever	
O.D.	inches	(mm)	inches	(mm)	lb./ft.	(kg/m)	inches³	(mm³)	x	psi	(Mpa)	=	Lb.In.		4'	6'
1 5/8"	1.660	42.16	0.111	2.82	1.84	2.74	0.1962	4.98	x	50000	345	=	9810	327	204	136
1 7/8"	1.900	48.26	0.120	3.05	2.28	3.39	0.2810	7.14	x	50000	345	=	14050	468	293	195
2 3/8"	2.375	60.33	0.130	3.30	3.12	4.64	0.4881	12.40	x	50000	345	=	24405	814	508	339
2 7/8"	2.875	73.03	0.160	4.06	4.64	6.91	0.8778	22.30	x	50000	345	=	43890	1463	914	610
3 1/2"	3.500	88.90	0.160	4.06	5.71	8.50	1.3408	34.06	x	50000	345	=	67042	2235	1397	931
4"	4.000	101.60	0.160	4.06	6.57	9.78	1.7820	45.26	x	50000	345	=	89098	2970	1856	1237
4 1/2"	4.500	114.30	0.160	4.14	7.42	11.04	2.2859	57.99	X	50000	345	=	114295	3810	5486	1587

Output:

Post Section Properties:

Post inside diameter: $ID_{post} := OD_{post} - 2 \cdot t_{post}$ $ID_{post} = 2.555\ in$

Post Area: $A_{post} := 0.785398 \cdot \left(OD_{post}^2 - ID_{post}^2\right)$ $A_{post} = 1.365\ in^2$

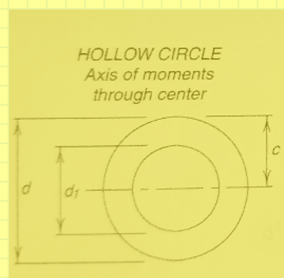
Post Unit Weight: $w_{post} := \gamma_{steel} \cdot A_{post}$ $w_{post} = 4.644\ plf$

Post centroid: $c_{post} := 0.5 \cdot OD_{post}$ $c_{post} = 1.438\ in$

Post Moment of Inertial: $I_{post} := 0.049087 \cdot \left(OD_{post}^4 - ID_{post}^4\right)$ $I_{post} = 1.262\ in^4$

Post Section Modulus: $S_{post} := \dfrac{I_{post}}{c_{post}}$ $S_{post} = 0.878\ in^3$

Post Plastic Section Modulus: $Z_{post} := \dfrac{OD_{post}^3 - ID_{post}^3}{6}$ $Z_{post} = 1.181\ in^3$



$A = \dfrac{\pi(d^2 - d_1^2)}{4} = .785398\ (d^2 - d_1^2)$

$c = \dfrac{d}{2}$

$I = \dfrac{\pi(d^4 - d_1^4)}{64} = .049087\ (d^4 - d_1^4)$

$S = \dfrac{\pi(d^4 - d_1^4)}{32\,d} = .098175\ \dfrac{d^4 - d_1^4}{d}$

$r = \dfrac{\sqrt{d^2 + d_1^2}}{4}$

$Z = \dfrac{d^3}{6} - \dfrac{d_1^3}{6}$

Rail Section Properties:

Rail inside diameter: $ID_{rail} := OD_{rail} - 2 \cdot t_{rail}$ $ID_{rail} = 1.438\ in$

Rail Area: $A_{rail} := 0.785398 \cdot \left(OD_{rail}^2 - ID_{rail}^2\right)$ $A_{rail} = 0.54\ in^2$

Rail Unit Weight: $w_{rail} := \gamma_{steel} \cdot A_{rail}$ $w_{rail} = 1.838\ plf$

Rail centroid: $c_{rail} := 0.5 \cdot OD_{rail}$ $c_{rail} = 0.83\ in$

Rail Moment of Inertial: $I_{rail} := 0.049087 \cdot \left(OD_{rail}^4 - ID_{rail}^4\right)$ $I_{rail} = 0.163\ in^4$

Rail Section Modulus: $S_{rail} := \dfrac{I_{rail}}{c_{rail}}$ $S_{rail} = 0.196\ in^3$

Rail Plastic Section Modulus: $Z_{rail} := \dfrac{OD_{rail}^3 - ID_{rail}^3}{6}$ $Z_{rail} = 0.267\ in^3$

Post concentrated live load applied at top rail: $P_{post\_LL} := P_{LL} + w_{LL} \cdot L_{spc} = 0.5\ kip$ $P_{post\_LL} = 0.5\ kip$ *AASHTO Eqn. 13.8.2-1*

Post moment loading from live load: $M_{post\_LL} := P_{post\_LL} \cdot H_{post} = 21000\ lbf \cdot in$ $M_{post\_LL} = 21000\ lbf \cdot in$ *Post treated as cantilevered beam*

Post shear from live load: $V_{post\_LL} := P_{post\_LL}$ $V_{post\_LL} = 0.5\ kip$

Rail moment from live load applied: $M_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}^{\ 2}}{8} + \frac{P_{LL} \cdot L_{spc}}{4}$ $M_{rail\_LL} = 7418.377\ lbf \cdot in$

Rail treated as simply supported beam with vertical and horizontal live loads combined into resultant direction.

Rail moment from dead load: $M_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}^{\ 2}}{8} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}^{\ 2}}{8}$ $M_{rail\_DL} = 144.615\ lbf \cdot in$

Rail shear from live load: $V_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}}{2} + \frac{P_{LL}}{2}$ $V_{rail\_LL} = 0.312\ kip$

Rail shear from dead load: $V_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}}{2} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}}{2}$ $V_{rail\_DL} = 0.008\ kip$

Factored Shear Load on Post: $V_{post\_u} := \gamma_{PL} \cdot V_{post\_LL}$ $V_{post\_u} = 0.875\ kip$

Factored Moment Load on Post: $M_{post\_u} := \gamma_{PL} \cdot M_{post\_LL}$ $M_{post\_u} = 36750\ lbf \cdot in$

AASHTO load factors used instead of ASCE load factors found in AISC and ACI. This is acceptable as it is more conservative.

Factored Shear Load on Rail: $V_{rail\_u} := \gamma_{PL} \cdot V_{rail\_LL} + \gamma_{DL} \cdot V_{rail\_DL}$ $V_{rail\_u} = 0.556\ kip$

Factored Moment Load on Rail: $M_{rail\_u} := \gamma_{PL} \cdot M_{rail\_LL} + \gamma_{DL} \cdot M_{rail\_DL}$ $M_{rail\_u} = 13162.928\ lbf \cdot in$

Vertical dead load was combined directly with live load resultant since it was so small compared to the live load.

Post Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area: $A_g := A_{post}$ $A_g = 1.365\ in^2$

Distance from Max to 0 Shear: $L_v := H_{post}$ $L_v = 42\ in$

Critical Strength for Shear:

$$F_{cr} := min\left(0.58 \cdot F_y, \max\left(\frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{post}}}\left(\frac{OD_{post}}{t_{post}}\right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left(\frac{OD_{post}}{t_{post}}\right)^{\frac{3}{2}}}\right)\right)$$

$F_{cr} = 29\ ksi$ *AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3*

Factored nominal shear resistance: $\phi V_n := \phi_v \cdot 0.5\ F_{cr} \cdot A_g$ $\phi V_n = 19.788\ kip$ *AASHTO Eqn. 6.12.1.2.3c-1*

Post Shear Check: $\frac{\phi V_n}{V_{post\_u}} = 22.615$

$Post\_Shear\_Check :=$ if $\frac{\phi V_n}{V_{post\_u}} \geq 1.0$
"Post shear strength is satisfactory."
else
"Post is not satisfactory."

$Post\_Shear\_Check =$ "Post shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section: $Check\_Compact :=$ if $\frac{OD_{post}}{t_{post}} \leq \frac{0.07 \cdot E_s}{F_y}$
"Section is compact. Local buckling does not apply."
else
"Section is not compact. Check wall slenderness."

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

$Check\_Compact =$ "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance: $\phi M_n := \phi_f \cdot F_y \cdot Z_{post}$ $\phi M_n = 59.038\ kip \cdot in$ *AASHTO Eqn. 6.12.2.2.3-1*

Post Flexural Check: $\frac{\phi M_n}{M_{post\_u}} = 1.606$

$Post\_Flex\_Check :=$ if $\frac{\phi M_n}{M_{post\_u}} \geq 1.0$
‖ "Post flexural strength is satisfactory."
else
‖ "Post is not satisfactory."

$Post\_Flex\_Check =$ "Post flexural strength is satisfactory."

Rail Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area: $A_g := A_{rail}$ $\quad A_g = 0.54\ in^2$

Distance from Max to 0 Shear: $L_v := \frac{L_{spc}}{2}$ $\quad L_v = 36\ in$

Critical Strength for Shear:

$$F_{cr} := min\left(0.58 \cdot F_y, \max\left(\frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{rail}}}\left(\frac{OD_{rail}}{t_{rail}}\right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left(\frac{OD_{rail}}{t_{rail}}\right)^{\frac{3}{2}}}\right)\right)$$

$F_{cr} = 29\ ksi$ *AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3*

Factored Nominal Shear Resistance: $\phi V_n := \phi_v \cdot 0.5\ F_{cr} \cdot A_g$ $\quad \phi V_n = 7.832\ kip$ *AASHTO Eqn. 6.12.1.2.3c-1*

Rail Shear Check: $\frac{\phi V_n}{V_{rail\_u}} = 14.08$

$Rail\_Shear\_Check :=$ if $\frac{\phi V_n}{V_{rail\_u}} \geq 1.0$
‖ "Rail shear strength is satisfactory."
else
‖ "Rail is not satisfactory."

$Rail\_Shear\_Check =$ "Rail shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

$Check\_Compact :=$ if $\frac{OD_{rail}}{t_{rail}} \leq \frac{0.07 \cdot E_s}{F_y}$
‖ "Section is compact. Local buckling does not apply."
else
‖ "Section is not compact. Check wall slenderness."

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

$Check\_Compact =$ "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance: $\phi M_n := \phi_f \cdot F_y \cdot Z_{rail}$ $\quad \phi M_n = 13.339\ kip \cdot in$ *AASHTO Eqn. 6.12.2.2.3-1*

Post Flexural Check: $\frac{\phi M_n}{M_{rail\_u}} = 1.013$

$Rail\_Flex\_Check :=$ if $\frac{\phi M_n}{M_{rail\_u}} \geq 1.0$
‖ "Rail flexural strength is satisfactory."
else
‖ "Rail is not satisfactory."

$Rail\_Flex\_Check =$ "Rail flexural strength is satisfactory."

Confirming that Wind Loading Doesn't Control:

Per last paragraph of AASHTO 13.8.2, the wind load on the chain link fence is not applied simultaneously with the live load.

Uniform wind load on post: $w_{post\_wind} := f_{wind} \cdot L_{spc}$ $w_{post\_wind} = 90\ plf$

Design moment from wind on post: $M_{post\_wind\_u} := \gamma_{WS} \cdot \frac{w_{post\_wind} \cdot H_{post}^{2}}{2}$ $M_{post\_wind\_u} = 6615\ lbf \cdot in$

$M_{post\_u} = 36750\ lbf \cdot in$ <- LL controls

Design shear from wind on post: $V_{post\_wind\_u} := \gamma_{WS} \cdot w_{post\_wind} \cdot H_{post}$ $V_{post\_wind\_u} = 0.315\ kip$

$V_{post\_u} = 0.875\ kip$ <- LL controls

Uniform wind on rail: $w_{rail\_wind} := f_{wind} \cdot \frac{H_{post}}{2}$ $w_{rail\_wind} = 26.25\ plf$

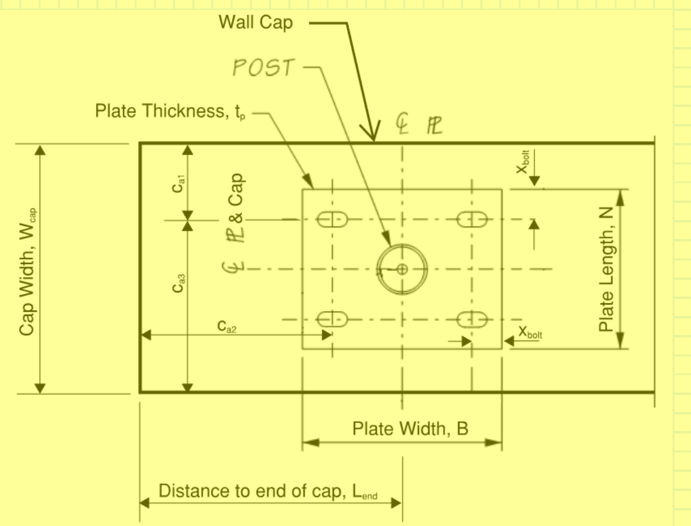
Design moment from wind on rail: $M_{rail\_wind\_u} := \gamma_{WS} \cdot \frac{w_{rail\_wind} \cdot L_{spc}^{2}}{8}$ $M_{rail\_wind\_u} = 1417.5\ lbf \cdot in$

$M_{rail\_u} = 13162.928\ lbf \cdot in$ <- LL controls

Design shear from wind on rail: $V_{rail\_wind\_u} := \gamma_{WS} \cdot w_{rail\_wind} \cdot \frac{L_{spc}}{2}$ $V_{rail\_wind\_u} = 0.079\ kip$

$V_{rail\_u} = 0.556\ kip$ <- LL controls

Base Plate Design - Line Post w/ Axial Compression



Given: Plans

Cap width:	$W_{cap} := 15.63\ in$
Distance from post to end of cap:	$L_{end} := 72\ in$
Plate thickness:	$t_p := 0.5\ in$
Plate length (perpendicular to fence):	$N_{plate} := 8\ in$
Plate width (parallel to fence):	$B_{plate} := 10\ in$
Compressive Strength of Concrete:	$f'_c := 4\ ksi$
Side clearance to anchor bolts:	$x_{bolt} := 1.5\ in$
Base plate steel yield strength:	$F_{y\_plate} := 36\ ksi$
Number of rails:	$n_{rail} := 2$

Output:

Plate Area:	$A_{plate} := N_{plate} \cdot B_{plate}$	$A_{plate} = 80\ in^2$	
Distance from bolt to near face of cap:	$c_{a1} := \frac{1}{2}\left(W_{cap} - N_{plate}\right) + x_{bolt}$	$c_{a1} = 5.315\ in$	
Distance from outside bolt to end of cap:	$c_{a2} := L_{end} - \frac{B_{plate}}{2} + x_{bolt}$	$c_{a2} = 68.5\ in$	
Distance from bolt to far face of cap:	$c_{a3} := W_{cap} - c_{a1}$	$c_{a3} = 10.315\ in$	
Bearing Area taken to Be Same as Plate Area:	$A_{bearing} := A_{plate}$	$A_{bearing} = 80\ in^2$	Conservatively setting bearing area to the same as the plate.
Max allowed bearing pressure:	$f_{pu\_max} := \phi_{brg} \cdot min\left(0.85 \cdot f'_c \cdot \sqrt{\frac{A_{bearing}}{A_{plate}}}, 1.7 \cdot f'_c\right)$	$f_{pu\_max} = 2.04\ ksi$	ACI Tbl. 14.5.6.1
Max allowed bearing pressure line:	$q_{max} := f_{pu\_max} \cdot B_{plate}$	$q_{max} = \left(2.448 \cdot 10^5\right) \frac{lbf}{ft}$	
Post dead load on plate	$P_{post\_DL} := w_{post} \cdot H_{post}$	$P_{post\_DL} = 0.016\ kip$	
Rail dead load on plat:	$P_{rail\_DL} := n_{rail} \cdot 2 \cdot V_{rail\_DL}$	$P_{rail\_DL} = 0.032\ kip$	
Factored vertical load on plate:	$P_u := \gamma_{DL} \cdot \left(P_{post\_DL} + P_{rail\_DL}\right)$	$P_u = 0.06\ kip$	
Minimum length of area of bearing:	$Y_{min} := \frac{P_u}{q_{max}}$	$Y_{min} = 0.003\ in$	AISC DG#1 Eqn. 3.3.3
Critical eccentricity distance:	$e_{crit} := \frac{N_{plate}}{2} - \frac{Y_{min}}{2}$	$e_{crit} = 3.999\ in$	AISC DG#1 Eqn. 3.3.7
Eccentricity of loading:	$e_{loading} := \frac{M_{post\_u}}{P_u}$	$e_{loading} = 607.565\ in$	AISC DG#1 Eqn. 3.3.6

Small moment check:

$Small\_Moment\_Check :=$ if $e_{loading} \le e_{crit}$
  "Moment is small, no need for anchor bolts."
else
  "Moment is large, need anchor bolts."

$Small\_Moment\_Check =$ "Moment is large, need anchor bolts."

Distance from bolt to center of post: $f_{dim} := \frac{N_{plate}}{2} - x_{bolt}$ $f_{dim} = 2.5\ in$

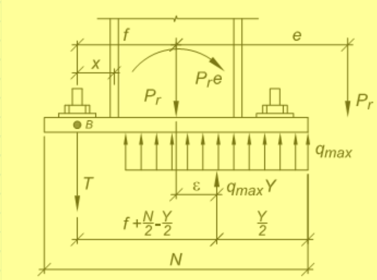


Fig. 3.4.1. Base plate with large moment.

Plate dimension check

$$Plate\_Dim\_Check := \text{if } \left(f_{dim}+\frac{N_{plate}}{2}\right)^2 \geq \frac{2 \cdot P_u \cdot \left(e_{loading}+f_{dim}\right)}{q_{max}}$$

"Plate dimensions are OK."

else

"Plate needs to be longer and/or wider."

$Plate\_Dim\_Check =$ "Plate dimensions are OK."

Length of bearing area centered at the eccentricity of this loading:

$$Y_{loading} := \left(f_{dim}+\frac{N_{plate}}{2}\right) - \sqrt{\left(f_{dim}+\frac{N_{plate}}{2}\right)^2 - \frac{2 \cdot P_u \cdot \left(e_{loading}+f_{dim}\right)}{q_{max}}}$$

$Y_{loading} = 0.285\ \boldsymbol{in}$

AISC DG#1 Eqn. 3.4.3

Required tensile resistance in anchor rods:

$$T_u := q_{max} \cdot Y_{loading} - P_u$$

$T_u = 5.744\ \boldsymbol{kip}$ *AISC DG#1 Eqn. 3.4.2*

Find minimum required thickness for plate based on bending at bearing interface:

Find bearing bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$m_{plate} := \frac{N_{plate} - 0.8\ OD_{post}}{2}$$

$m_{plate} = 2.85\ \boldsymbol{in}$

Calculating minimum thickness based on bearing:

$$t_{p\_brng\_req} := \text{if } Y_{loading} \geq m_{plate}$$

$$1.5 \cdot m_{plate} \cdot \sqrt{\frac{f_{pu\_max}}{F_{y\_plate}}}$$

else

$$2.11 \cdot \sqrt{\frac{f_{pu\_max} \cdot Y_{loading} \cdot \left(m_{plate} - \frac{Y_{loading}}{2}\right)}{F_{y\_plate}}}$$

$t_{p\_brng\_req} = 0.441\ \boldsymbol{in}$

AISC DG#1 Eqns. 3.3.14a-2 & 3.3.15a-2

Find minimum required thickness for plate based on bending at tension interface:

Find tension bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$x_{ten} := f_{dim} - \frac{0.8 \cdot OD_{post}}{2}$$

$x_{ten} = 1.35\ \boldsymbol{in}$

Calculating minimum thickness based on tension:

$$t_{p\_ten\_req} := 2.11 \cdot \sqrt{\frac{T_u \cdot x_{ten}}{B_{plate} \cdot F_{y\_plate}}}$$

$t_{p\_ten\_req} = 0.31\ \boldsymbol{in}$ *AISC DG#1 Eqn. 3.4.7a*

Controlling minimum required base plate thickness:

$$t_{p\_req} := \max\left(t_{p\_brng\_req}, t_{p\_ten\_req}\right)$$

$t_{p\_req} = 0.441\ \boldsymbol{in}$

Check chosen plate thickness:

$$Plate\_Thick\_Check := \text{if } t_p \geq t_{p\_req}$$

"Chosen plate thickness is adequate."

else

"Need a thicker plate."

$Plate\_Thick\_Check =$ "Chosen plate thickness is adequate."

Pipe to Plate Fillet Weld Connection Design

Given:

Minimum Fillet Weld Size: $w_{min} := \frac{1}{8}\ in$ *Min fillet weld size based on AISC Table J2-4*

Chosen fillet weld size $w := \frac{5}{16}\ in$

Weld material: $F_{EXX} := 70\ ksi$

Output:

Welded Connection to Base Plate Design:

Gross Length of Weld is Post Perimeter: $L_g := \pi \cdot OD_{post}$ $L_g = 9.032\ in$

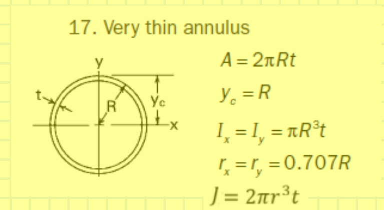
Effective Length of Weld: $L_w := L_g - 2 \cdot w$ $L_w = 8.407\ in$

Effective Throat Thickness: $t_e := min\left(w \cdot \sin(45\ deg), \frac{L_w}{4}\right)$ $t_e = 0.221\ in$ *AISC, Sect. J2, Pts. 2a & 2b*

Area of Weld: $A_w := L_w \cdot t_e$ $A_w = 1.858\ in^2$ *AISC, Sect. J2, Pts. 2a*

Moment of Inertia of circular fillet weld: $I_w := \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e$ $I_w = 2.062\ in^4$

Polar moment of Inertia of circular fillet weld: $J_w := 2\ \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e$ $J_w = 4.124\ in^4$



Determine design strength of weld:

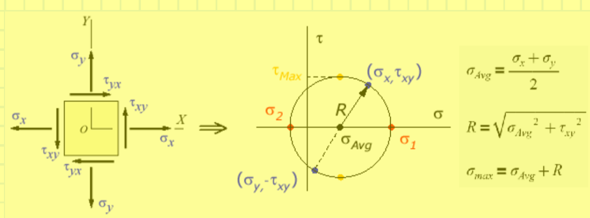
Nominal strength of weld metal: $F_w := \phi_{fw} \cdot 0.6 \cdot F_{EXX}$ $F_w = 31.5\ ksi$ *AISC, Tbl. J2.5*

Normal stress caused by bending moment: $\sigma_b := \dfrac{M_{post\_u} \cdot \left(\frac{OD_{post}}{2}\right)}{I_w}$ $\sigma_b = 25.619\ ksi$

$$\sigma = \frac{M}{S} = \frac{M \cdot c}{I}$$

Stress caused by shearing force: $\tau_v := \dfrac{V_{post\_u}}{A_w}$ $\tau_v = 0.471\ ksi$

Resultant stress in weld from loading: $\sigma_{max} := \frac{\sigma_b}{2} + \sqrt{\left(\frac{\sigma_b}{2}\right)^2 + \tau_v^{\ 2}}$ $\sigma_{max} = 25.627\ ksi$

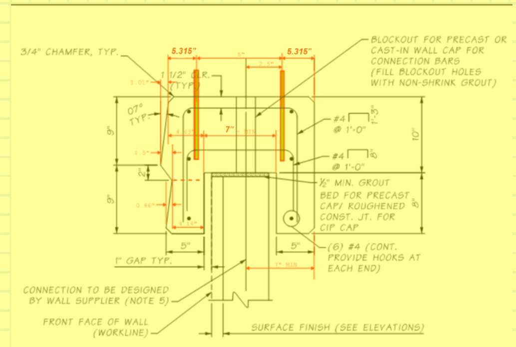


Check of weld thickness:

$Weld\_Design\_Check :=$ if $F_w \geq \sigma_{max}$
    "Chosen weld size is sufficient."
else
    "Need bigger fillet weld."

$Weld\_Design\_Check =$ "Chosen weld size is sufficient."

Anchor Bolt Connection Design



WALL CAP TYPICAL DETAIL

Given:

Number of anchor bolts resisting loads: $n_{ab} := 2$ *Only one side's bolts resist tension or shear.*

Bolts are specified as ASTM F1554 and Grade A36

Bolt diameter: $d_{ab} := \frac{5}{8}\ in$ *Plans*

Bolt area: $A_b := 0.307\ in^2$ *AISC Tbl. 7-18*

Bolt nominal yield stress strength: $F_{y\_bolt} := 36\ ksi$ *AISC Tbl. 2-3*

Bolt nominal ultimate tensile stress strength: $F_{u\_bolt} := 58\ ksi$

Bolt embedment: $h_{ef} := 5\ in$ *Plans*

Output:

Tension anchor bolt spacing: $s_1 := \frac{B_{plate} - 2 \cdot x_{bolt}}{n_{ab} - 1}$ $s_1 = 7\ in$

Bolt nominal tensile stress strength: $F_{nt} := 0.75 \cdot F_{u\_bolt}$ $F_{nt} = 43.5\ ksi$ *AISC Tbl. J3.2*

Bolt nominal shear stress strength: $F_{nv} := 0.40 \cdot F_{u\_bolt}$ $F_{nv} = 23.2\ ksi$ *AISC Tbl. J3.2, assuming threads within shear plane*

Ultimate tension load on one anchor bolt: $T_{u\_ab} := \frac{T_u}{n_{ab}}$ $T_{u\_ab} = 2.872\ kip$

Required shear stress on one bolt: $f_v := \frac{V_{post\_u}}{n_{ab} \cdot A_b}$ $f_v = 1.425\ ksi$

Bolt modified nominal tensile stress strength, modified for effects of shear stress: $F_{nt}' := min\left(F_{nt}, 1.3 \cdot F_{nt} - \frac{F_{nt}}{\phi_{ab} \cdot F_{nv}} \cdot f_v\right)$ $F_{nt}' = 43.5\ ksi$ *AISC Eqn. J3-3a*

Bolt factored tensile resistance: $\phi R_{n\_bolt} := \phi_{ab} \cdot F_{nt}' \cdot A_b$ $\phi R_{n\_bolt} = 10.016\ kip$ *AISC Eqn. J3-2*

Check of bolt tensile stress:

$Bolt\_Tensile\_Check :=$ if $\phi R_{n\_bolt} \ge T_{u\_ab}$
  "Bolt is satisfactory."
else
  "Bolt is no good."

$Bolt\_Tensile\_Check =$ "Bolt is satisfactory."

Continuing Anchor Bolt Connection Design per ACI 318

Outside diameter of anchor: $d_a := d_{ab}$ $d_a = 0.625\ in$

Critical edge distance for adhesive anchors: $c_{ac} := 2\ h_{ef}$ $c_{ac} = 10\ in$ *ACI 17.7.6*

Steel strength of anchor in tension (ACI 17.4.1)

Steel tension strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in tension (ACI 17.4.2)

Check bolt group action for tension concrete breakout: *ACI 17.2.1.1*

$Group\_Tension\_Breakout\_Check :=$ if $s_1 \le 3 \cdot h_{ef}$
  "Bolts act in group."
else
  "Bolts act singly."

$Group\_Tension\_Breakout\_Check =$ "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Nco} := 9 \cdot h_{ef}^2$ $A_{Nco} = 225\ in^2$ *ACI Eqn. 17.4.2.1c*

Actual projected influence area for bolt(s): $A_{Nc} := min\left(\left(c_{a1} + 1.5 \cdot h_{ef}\right) \cdot \left(1.5 \cdot h_{ef} + min\left(s_1, 3 \cdot h_{ef}\right) + min\left(1.5 \cdot h_{ef}, c_{a2}\right)\right), n_{ab} \cdot A_{Nco}\right)$ $A_{Nc} = 281.93\ in^2$ *ACI Fig. R17.4.2.1*

Concrete tension breakout strength coefficient: $k_c := 17$ *Value of 17 for post-installed anchors, per ACI 17.4.2.2*

Basic concrete tension breakout strength for single anchor: $N_b := k_c \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lbf$ $N_b = 12.021\ kip$ *ACI Eqn. 17.4.2.2a*

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_N} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.4.2.4*

Factor for anchor bolts near an edge: $\Psi_{ed\_N} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a1}}{1.5 \cdot h_{ef}}\right)$ $\Psi_{ed\_N} = 0.913$ *ACI Eqn. 17.4.2.5b*

Factor for anchor bolts in un-cracked concrete: $\Psi_{c\_N} := 1.4$ *Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.* *ACI 17.4.2.6*

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement: $\Psi_{cp\_N} := min\left(1.0, \max\left(\frac{c_{a1}}{c_{ac}}, \frac{1.5 \cdot h_{ef}}{c_{ac}}\right)\right)$ $\Psi_{cp\_N} = 0.75$ *ACI Eqn. 17.4.2.7b*

Nominal concrete tension breakout strength: $\phi N_{cbg} := \phi_{adh} \cdot \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ec\_N} \cdot \Psi_{ed\_N} \cdot \Psi_{c\_N} \cdot \Psi_{cp\_N} \cdot N_b$ $\phi N_{cbg} = 9.382\ kip$ *ACI Eqn. 17.4.2.1b*

Check of concrete tension breakout failure:

$$Concrete\_Tension\_Breakout\_Check := \begin{cases} \text{"Bolt is satisfactory."} & \text{if } \phi N_{cbg} \geq n_{ab} \cdot T_{u\_ab} \\ \text{"Bolt is no good."} & \text{else} \end{cases}$$

$Concrete\_Tension\_Breakout\_Check =$ "Bolt is satisfactory."

Pullout strength cast-in, post-installed expansion, or undercut anchor in tension (ACI 17.4.3)

Proposed anchors are post-installed adhesive, not headed studs or bolts, expansion anchors, or undercut anchors; so, no check is required.

Concrete side-face blowout strength of headed anchor in tension (ACI 17.4.4)

Proposed anchors are post-installed adhesive, not headed studs or bolts; so, no check is required.

Bond strength of adhesive anchor in tension (ACI 17.4.5)

Minimum bond stress for HY 200 Epoxy per HILTI ESR-3187:

$$\tau_{uncr\_HY\_200} := 0.65 \cdot \left(\frac{f'_c}{2500\ psi}\right)^{0.1} \cdot 2220\ psi = 1512.441\ psi$$

Per HILTI ESR-3187 Table 14, basic un-cracked bond strength is 2,220 psi; this value is factored by a straight 0.65 for either wet or dry installation conditions and by a small boost from concrete strength higher than 2,500 psi

Minimum bond stress for HIT-RE 500 Epoxy per HILTI ESR-3814:

$$\tau_{uncr\_HIT\_RE\_500} := 0.65 \cdot \left(\frac{f'_c}{2500\ psi}\right)^{0.15} \cdot 2210\ psi = 1541.429\ psi$$

Per HILTI ESR-3814 Table 12, basic un-cracked bond strength is 2,210 psi. This value is based on diamond coring and roughening afterwards; it is lower than being hammer-drilled with carbide bit. The socket must be roughened if coring with a diamond bit; this should be written on the plans. Factors are a straight 0.65 reduction factor independent of wet or dry concrete conditions during installation and a small boost for using concrete higher than 2,500 psi. The smaller factor for cracked concrete is used since no supplementary rebar is being provided; this also matches with reduction factor below.

Minimum bond stress strength: $\tau_{uncr} := min\left(\tau_{uncr\_HY\_200}, \tau_{uncr\_HIT\_RE\_500}\right)$ $\tau_{uncr} = 1512.441\ psi$

Distance to edge of project influence area: $c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100\ psi}}$ $c_{Na} = 7.329\ in$ *ACI Eqn. 17.4.5.1d*

Check if anchor bolts act in group for bond failure: *ACI 17.2.1.1*

$$Group\_Bond\_Failure\_Check := \begin{cases} \text{"Bolts act in group."} & \text{if } s_1 \leq 2\ c_{Na} \\ \text{"Bolts act singly."} & \text{else} \end{cases}$$

$Group\_Bond\_Failure\_Check =$ "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Nao} := (2\ c_{Na})^2$ $A_{Nao} = 214.835\ in^2$ *ACI Eqn. 17.4.5.1c*

Actual projected influence area for bolt(s): $A_{Na} := min\left(\left(c_{Na} + min\left(s_1, 2 \cdot c_{Na}\right) + min\left(c_{Na}, c_{a2}\right)\right) \cdot \left(c_{a1} + c_{Na}\right), n_{ab} \cdot A_{Nao}\right)$ $A_{Na} = 273.826\ in^2$ *ACI Fig. R17.4.5.1*

Basic bond strength of adhesive anchor: $N_{ba} := \tau_{uncr} \cdot \pi \cdot d_a \cdot h_{ef}$ $N_{ba} = 14.848\ kip$ *ACI Eqn. 17.4.5.2*

Concrete is not light weight; so, lambda-a is set to 1.0; per ACI 17.4.5.2, un-cracked bond stress may be used.

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_Na} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.4.5.3*

Factor for anchor bolts near an edge: $\Psi_{ed\_Na} := min\left(1.0, 0.7 + 0.3 \cdot \dfrac{c_{a1}}{c_{Na}}\right)$ $\Psi_{ed\_Na} = 0.918$ *ACI Eqn. 17.4.5.4b*

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement: $\Psi_{cp\_Na} := min\left(1.0, \max\left(\dfrac{c_{a1}}{c_{ac}}, \dfrac{c_{Na}}{c_{ac}}\right)\right)$ $\Psi_{cp\_Na} = 0.733$ *ACI Eqn. 17.4.5.5b*

Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

Nominal bond strength of the adhesive anchor(s): $\phi N_{ag} := \phi_{adh} \cdot \dfrac{A_{Na}}{A_{Nao}} \cdot \Psi_{ec\_Na} \cdot \Psi_{ed\_Na} \cdot \Psi_{cp\_Na} \cdot N_{ba}$ $\phi N_{ag} = 8.272\ kip$ *ACI Eqn. 17.4.5.1b*

Check of bolt bond stress failure:

$Bond\_Stress\_Check :=$ if $\phi N_{ag} \ge n_{ab} \cdot T_{u\_ab}$
  "Bolt is satisfactory."
else
  "Bolt is no good."

$Bond\_Stress\_Check =$ "Bolt is satisfactory."

Steel strength of anchor in shear (17.5.1)

Steel shear strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in shear (17.5.2)

Check bolt group action for shear concrete breakout: *ACI 17.2.1.1*

$Group\_Shear\_Breakout\_Check :=$ if $s_1 \le 3 \cdot c_{a1}$
  "Bolts act in group."
else
  "Bolts act singly."

$Group\_Shear\_Breakout\_Check =$ "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Vco} := 4.5 \cdot {c_{a1}}^2$ $A_{Vco} = 127.122\ in^2$ *ACI Eqn. 17.5.2.1c*

Actual projected influence area for bolt(s): $A_{Vc} := min\left(1.5 \cdot c_{a1} \cdot \left(1.5 \cdot c_{a1} + min\left(s_1, 3 \cdot c_{a1}\right) + min\left(1.5 \cdot c_{a1}, c_{a2}\right)\right), n_{ab} \cdot A_{Vco}\right)$ $A_{Vc} = 182.929\ in^2$ *ACI Fig. R17.5.2.1b*

Load bearing length: $l_e := h_{ef}$ $l_e = 5\ in$ *ACI 17.5.2.2*

Basic concrete breakout strength in shear for single anchor:

$$V_b := min\left(\left(7\left(\frac{l_e}{d_a}\right)^{0.2} \cdot \sqrt{\frac{d_a}{in}}\right) \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5}, 9 \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5}\right) \cdot lbf \qquad V_b = 6.5\ kip$$

Concrete is not light weight; so, lambda-a is set to 1.0. *ACI Eqns. 17.5.2.2a & 17.5.2.2b*

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_V} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.5.2.5*

Factor for anchor bolts near an edge: $\Psi_{ed\_V} := min\left(1.0, 0.7 + 0.3 \cdot \dfrac{c_{a2}}{1.5 \cdot c_{a1}}\right)$ $\Psi_{ed\_V} = 1$ *ACI Eqns. 17.5.2.6a & 17.5.2.6b*

Factor for anchor bolts in un-cracked concrete: $\Psi_{c\_V} := 1.4$ *Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.* *ACI 17.5.2.7*

Factor for small embedment $\Psi_{h\_V} := min\left(1.0, \sqrt{\frac{1.5 \cdot c_{a1}}{h_{ef}}}\right)$ $\Psi_{h\_V} = 1$ *ACI Eqn. 17.5.2.8*

Nominal concrete shear breakout strength: $\phi V_{cbg} := \phi_{adh} \cdot \frac{A_{Vc}}{A_{Vco}} \cdot \Psi_{ec\_V} \cdot \Psi_{ed\_V} \cdot \Psi_{c\_V} \cdot \Psi_{h\_V} \cdot V_b$ $\phi V_{cbg} = 8.512\ kip$ *ACI Eqn. 17.5.2.1b*

Check of concrete shear breakout failure:

$Concrete\_Shear\_Breakout\_Check :=$ if $\phi V_{cbg} \geq V_{post\_u}$
  "Bolt is satisfactory."
else
  "Bolt is no good."

$Concrete\_Shear\_Breakout\_Check =$ "Bolt is satisfactory."

Concrete pryout strength of anchor in shear (17.5.3)

Basic concrete pryout strength of a single anchor in shear: $\phi N_{cpg} := min\left(\phi N_{ag}, \phi N_{cbg}\right)$ $\phi N_{cpg} = 8.272\ kip$ *ACI 17.5.3.1*

Concrete pryout strength in shear coefficient:

$k_{cp} :=$ if $h_{ef} < 2.5\ in$
  1.0
else
  2.0

$k_{cp} = 2$ *ACI 17.5.3.1*

Nominal concrete pryout strength of anchor(s) in shear: $\phi V_{cpg} := k_{cp} \cdot \phi N_{cpg}$ $\phi V_{cpg} = 16.545\ kip$ *ACI Eqn. 17.5.3.1b*

Check of concrete pryout strength in shear:

$Concrete\_Shear\_Pryout\_Check :=$ if $\phi V_{cpg} \geq V_{post\_u}$
  "Bolt is satisfactory."
else
  "Bolt is no good."

$Concrete\_Shear\_Pryout\_Check =$ "Bolt is satisfactory."

Vertical Interior Post and Horizontal Rail Design

Given:

Post Height: $H_{post} := 66\ \boldsymbol{in}$ *Plans*

Step Height: $H_{step} := 24\ \boldsymbol{in}$

Post and Rail Yield Strength: $F_y := 50\ \boldsymbol{ksi}$ *ASTM F1043*

Post and Rail Modulus of Elasticity: $E_s := 29000\ \boldsymbol{ksi}$

Post and Rail Ultimate Strength: $F_u := 58\ \boldsymbol{ksi}$

Properties for ASTM F1043 IC 2-7/8" Pipe for Interior Posts

Post OD: $OD_{post} := 2.875\ \boldsymbol{in}$

Post Thickness: $t_{post} := 0.160\ \boldsymbol{in}$

Properties for ASTM F1043 IC 1-5/8" Pipe for Rails

Rail OD: $OD_{rail} := 1.660\ \boldsymbol{in}$

Rail Thickness: $t_{rail} := 0.111\ \boldsymbol{in}$

Design Point Live Load $P_{LL} := 200\ \boldsymbol{lbf}$ *AASHTO 13.8.2*

Design Uniform Live Load $w_{LL} := 50\ \boldsymbol{plf}$ *AASHTO 13.8.2*

Post spacing: $L_{spc} := 4\ \boldsymbol{ft}$ *Plans*

Weight of chain link fence: $f_{clf} := 0.48\ \boldsymbol{psf}$

Design wind load from chain link fence: $f_{wind} := 15\ \boldsymbol{psf}$ *AASHTO 13.8.2*

Load Factors (AASHTO Tbl. 3.4.1-1):

PL Load Factor: $\gamma_{PL} := 1.75$

DC Load Factor: $\gamma_{DL} := 1.25$

WS Load Factor: $\gamma_{WS} := 1.00$

Resistance Factors:

Steel Flexure (AASHTO 6.5.4.2): $\phi_f := 1.00$

Steel Shear (AASHTO 6.5.4.2): $\phi_v := 1.00$

Tension, Yielding in Gross Section: $\phi_y := 0.95$

Bending (AISC F1): $\phi_b := 0.90$

Shear (AISC G1): $\phi_{v\_AISC} := 0.90$

Bearing (AISC DG#1): $\phi_{brg} := 0.60$

Fillet Weld (AISC Tbl. J2.5): $\phi_{fw} := 0.75$

Bolts (AISC J3.6 & J3.7): $\phi_{ab} := 0.75$

Adhesive Anchor Bolts (ACI 17.3.3, Condition B, Category 1): $\phi_{adh} := 0.65$

Steel weight density: $\gamma_{steel} := 490\ \boldsymbol{pcf}$

ASTM F1043 Group IC Electric Resistant Welded 50,000 psi yield steel pipe

Trade Reference	Decimal O.D. Equivalent		Pipe wall Thickness		Weight		Section Modulus		x	Min. Yield Strength		=	Max Bending Moment	Calculated Load (lbs)		
														10' Free	Cantilever	
O.D.	inches	(mm)	inches	(mm)	lb./ft.	(kg/m)	inches³	(mm³)	x	psi	(Mpa)	=	Lb.In.	Supported	4'	6'
1 5/8"	1.660	42.16	0.111	2.82	1.84	2.74	0.1962	4.98	x	50000	345	=	9810	327	204	136
1 7/8"	1.900	48.26	0.120	3.05	2.28	3.39	0.2810	7.14	x	50000	345	=	14050	468	293	195
2 3/8"	2.375	60.33	0.130	3.30	3.12	4.64	0.4881	12.40	x	50000	345	=	24405	814	508	339
2 7/8"	2.875	73.03	0.160	4.06	4.64	6.91	0.8778	22.30	x	50000	345	=	43890	1463	914	610
3 1/2"	3.500	88.90	0.160	4.06	5.71	8.50	1.3408	34.06	x	50000	345	=	67042	2235	1397	931
4"	4.000	101.60	0.160	4.06	6.57	9.78	1.7820	45.26	x	50000	345	=	89098	2970	1856	1237
4 1/2"	4.500	114.30	0.160	4.14	7.42	11.04	2.2859	57.99	X	50000	345	=	114295	3810	5486	1587

Output:

Post Section Properties:

Post inside diameter: $ID_{post} := OD_{post} - 2 \cdot t_{post}$ $ID_{post} = 2.555\ \boldsymbol{in}$

Post Area: $A_{post} := 0.785398 \cdot \left(OD_{post}^{\ 2} - ID_{post}^{\ 2}\right)$ $A_{post} = 1.365\ \boldsymbol{in}^2$

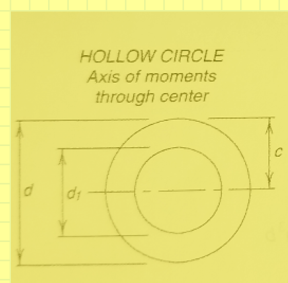
Post Unit Weight: $w_{post} := \gamma_{steel} \cdot A_{post}$ $w_{post} = 4.644\ \boldsymbol{plf}$

Post centroid: $c_{post} := 0.5 \cdot OD_{post}$ $c_{post} = 1.438\ \boldsymbol{in}$

Post Moment of Inertial: $I_{post} := 0.049087 \cdot \left(OD_{post}^{\ 4} - ID_{post}^{\ 4}\right)$ $I_{post} = 1.262\ \boldsymbol{in}^4$

Post Section Modulus: $S_{post} := \dfrac{I_{post}}{c_{post}}$ $S_{post} = 0.878\ \boldsymbol{in}^3$

Post Plastic Section Modulus: $Z_{post} := \dfrac{OD_{post}^{\ 3} - ID_{post}^{\ 3}}{6}$ $Z_{post} = 1.181\ \boldsymbol{in}^3$



$$A = \frac{\pi(d^2 - d_1^2)}{4} = .785398\,(d^2 - d_1^2)$$

$$c = \frac{d}{2}$$

$$I = \frac{\pi(d^4 - d_1^4)}{64} = .049087\,(d^4 - d_1^4)$$

$$S = \frac{\pi(d^4 - d_1^4)}{32d} = .098175\,\frac{d^4 - d_1^4}{d}$$

$$r = \frac{\sqrt{d^2 + d_1^2}}{4}$$

$$Z = \frac{d^3}{6} - \frac{d_1^3}{6}$$

Rail Section Properties:

Rail inside diameter: $ID_{rail} := OD_{rail} - 2 \cdot t_{rail}$ $ID_{rail} = 1.438\ \boldsymbol{in}$

Rail Area: $A_{rail} := 0.785398 \cdot \left(OD_{rail}^{\ 2} - ID_{rail}^{\ 2}\right)$ $A_{rail} = 0.54\ \boldsymbol{in}^2$

Rail Unit Weight: $w_{rail} := \gamma_{steel} \cdot A_{rail}$ $w_{rail} = 1.838\ \boldsymbol{plf}$

Rail centroid: $c_{rail} := 0.5 \cdot OD_{rail}$ $c_{rail} = 0.83\ \boldsymbol{in}$

Rail Moment of Inertial: $I_{rail} := 0.049087 \cdot \left(OD_{rail}^{\ 4} - ID_{rail}^{\ 4}\right)$ $I_{rail} = 0.163\ \boldsymbol{in}^4$

Rail Section Modulus: $S_{rail} := \dfrac{I_{rail}}{c_{rail}}$ $S_{rail} = 0.196\ \boldsymbol{in}^3$

Rail Plastic Section Modulus: $Z_{rail} := \dfrac{OD_{rail}^{\ 3} - ID_{rail}^{\ 3}}{6}$ $Z_{rail} = 0.267\ \boldsymbol{in}^3$

Post concentrated live load applied at high top rail: $P_{post\_LL\_H} := P_{LL} + w_{LL} \cdot \frac{L_{spc}}{2} = 0.3\ kip$ $\quad P_{post\_LL\_H} = 0.3\ kip$ *AASHTO Eqn. 13.8.2-1, modified for split top rails*

Post concentrated live load applied at low top rail: $P_{post\_LL\_L} := w_{LL} \cdot \frac{L_{spc}}{2} = 0.1\ kip$ $\quad P_{post\_LL\_L} = 0.1\ kip$

Post moment loading from live load: $M_{post\_LL} := P_{post\_LL\_H} \cdot H_{post} + P_{post\_LL\_L} \cdot \left(H_{post} - H_{step}\right)$ $\quad M_{post\_LL} = 24000\ lbf \cdot in$ *Post treated as cantilevered beam*

Post shear from live load: $V_{post\_LL} := P_{post\_LL\_H} + P_{post\_LL\_L}$ $\quad V_{post\_LL} = 0.4\ kip$

Rail moment from live load applied: $M_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}^{\ 2}}{8} + \frac{P_{LL} \cdot L_{spc}}{4}$ $\quad M_{rail\_LL} = 4097.056\ lbf \cdot in$ *Rail treated as simply supported beam with vertical and horizontal live loads combined into resultant direction.*

Rail moment from dead load: $M_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}^{\ 2}}{8} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}^{\ 2}}{8}$ $\quad M_{rail\_DL} = 75.793\ lbf \cdot in$

Rail shear from live load: $V_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}}{2} + \frac{P_{LL}}{2}$ $\quad V_{rail\_LL} = 0.241\ kip$

Rail shear from dead load: $V_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}}{2} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}}{2}$ $\quad V_{rail\_DL} = 0.006\ kip$

Factored Shear Load on Post: $V_{post\_u} := \gamma_{PL} \cdot V_{post\_LL}$ $\quad V_{post\_u} = 0.7\ kip$

Factored Moment Load on Post: $M_{post\_u} := \gamma_{PL} \cdot M_{post\_LL}$ $\quad M_{post\_u} = 42000\ lbf \cdot in$

AASHTO load factors used instead of ASCE load factors found in AISC and ACI. This is acceptable as it is more conservative.

Factored Shear Load on Rail: $V_{rail\_u} := \gamma_{PL} \cdot V_{rail\_LL} + \gamma_{DL} \cdot V_{rail\_DL}$ $\quad V_{rail\_u} = 0.43\ kip$

Factored Moment Load on Rail: $M_{rail\_u} := \gamma_{PL} \cdot M_{rail\_LL} + \gamma_{DL} \cdot M_{rail\_DL}$ $\quad M_{rail\_u} = 7264.59\ lbf \cdot in$

Vertical dead load was combined directly with live load resultant since it was so small compared to the live load.

Post Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area: $A_g := A_{post}$ $\quad A_g = 1.365\ in^2$

Distance from Max to 0 Shear: $L_v := H_{post}$ $\quad L_v = 66\ in$

Critical Strength for Shear:

$$F_{cr} := min\left(0.58 \cdot F_y, \max\left(\frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{post}}} \left(\frac{OD_{post}}{t_{post}}\right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left(\frac{OD_{post}}{t_{post}}\right)^{\frac{3}{2}}}\right)\right)$$

$F_{cr} = 29\ ksi$ *AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3*

Factored nominal shear resistance: $\phi V_n := \phi_v \cdot 0.5\ F_{cr} \cdot A_g$ $\quad \phi V_n = 19.788\ kip$ *AASHTO Eqn. 6.12.1.2.3c-1*

Post Shear Check: $\frac{\phi V_n}{V_{post\_u}} = 28.269$

$Post\_Shear\_Check :=$ if $\frac{\phi V_n}{V_{post\_u}} \ge 1.0$
 "Post shear strength is satisfactory."
else
 "Post is not satisfactory."

$Post\_Shear\_Check =$ "Post shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

$Check\_Compact :=$ if $\frac{OD_{post}}{t_{post}} \le \frac{0.07 \cdot E_s}{F_y}$
 "Section is compact. Local buckling does not apply."
else
 "Section is not compact. Check wall slenderness."

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

$Check\_Compact =$ "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance: $\phi M_n := \phi_f \cdot F_y \cdot Z_{post}$ $\phi M_n = 59.038$ ***kip • in*** *AASHTO Eqn. 6.12.2.2.3-1*

Post Flexural Check: $\dfrac{\phi M_n}{M_{post\_u}} = 1.406$

$$Post\_Flex\_Check := \text{if } \frac{\phi M_n}{M_{post\_u}} \ge 1.0 \;\Big\|\; \text{“Post flexural strength is satisfactory.”} \;\text{else}\; \Big\|\; \text{“Post is not satisfactory.”}$$

$Post\_Flex\_Check =$ “Post flexural strength is satisfactory.”

Rail Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area: $A_g := A_{rail}$ $A_g = 0.54$ ***in***^{2}

Distance from Max to 0 Shear: $L_v := \dfrac{L_{spc}}{2}$ $L_v = 24$ ***in***

Critical Strength for Shear:

$$F_{cr} := min\left(0.58 \cdot F_y, \max\left(\frac{1.6 \cdot E_s}{\sqrt{\dfrac{L_v}{OD_{rail}}}\left(\dfrac{OD_{rail}}{t_{rail}}\right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left(\dfrac{OD_{rail}}{t_{rail}}\right)^{\frac{3}{2}}}\right)\right)$$

$F_{cr} = 29$ ***ksi*** *AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3*

Factored Nominal Shear Resistance: $\phi V_n := \phi_v \cdot 0.5\ F_{cr} \cdot A_g$ $\phi V_n = 7.832$ ***kip*** *AASHTO Eqn. 6.12.1.2.3c-1*

Rail Shear Check: $\dfrac{\phi V_n}{V_{rail\_u}} = 18.199$

$$Rail\_Shear\_Check := \text{if } \frac{\phi V_n}{V_{rail\_u}} \ge 1.0 \;\Big\|\; \text{“Rail shear strength is satisfactory.”} \;\text{else}\; \Big\|\; \text{“Rail is not satisfactory.”}$$

$Rail\_Shear\_Check =$ “Rail shear strength is satisfactory.”

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

$$Check\_Compact := \text{if } \frac{OD_{rail}}{t_{rail}} \le \frac{0.07 \cdot E_s}{F_y} \;\Big\|\; \text{“Section is compact. Local buckling does not apply.”} \;\text{else}\; \Big\|\; \text{“Section is not compact. Check wall slenderness.”}$$

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

$Check\_Compact =$ “Section is compact. Local buckling does not apply.”

Factored Nominal Moment Resistance: $\phi M_n := \phi_f \cdot F_y \cdot Z_{rail}$ $\phi M_n = 13.339$ ***kip • in*** *AASHTO Eqn. 6.12.2.2.3-1*

Post Flexural Check: $\dfrac{\phi M_n}{M_{rail\_u}} = 1.836$

$$Rail\_Flex\_Check := \text{if } \frac{\phi M_n}{M_{rail\_u}} \ge 1.0 \;\Big\|\; \text{“Rail flexural strength is satisfactory.”} \;\text{else}\; \Big\|\; \text{“Rail is not satisfactory.”}$$

$Rail\_Flex\_Check =$ “Rail flexural strength is satisfactory.”

Confirming that Wind Loading Doesn't Control:

Per last paragraph of AASHTO 13.8.2, the wind load on the chain link fence is not applied simultaneously with the live load.

Uniform wind load on post: $w_{post\_wind} := f_{wind} \cdot L_{spc}$ $\quad w_{post\_wind} = 60\ plf$

Design moment from wind on post: $M_{post\_wind\_u} := \gamma_{WS} \cdot \dfrac{w_{post\_wind} \cdot H_{post}^2}{2}$ $\quad M_{post\_wind\_u} = 10890\ lbf \cdot in$

$M_{post\_u} = 42000\ lbf \cdot in$ <- LL controls

Design shear from wind on post: $V_{post\_wind\_u} := \gamma_{WS} \cdot w_{post\_wind} \cdot H_{post}$ $\quad V_{post\_wind\_u} = 0.33\ kip$

$V_{post\_u} = 0.7\ kip$ <- LL controls

Uniform wind on rail: $w_{rail\_wind} := f_{wind} \cdot \dfrac{H_{post}}{2}$ $\quad w_{rail\_wind} = 41.25\ plf$

Design moment from wind on rail: $M_{rail\_wind\_u} := \gamma_{WS} \cdot \dfrac{w_{rail\_wind} \cdot L_{spc}^2}{8}$ $\quad M_{rail\_wind\_u} = 990\ lbf \cdot in$

$M_{rail\_u} = 7264.59\ lbf \cdot in$ <- LL controls

Design shear from wind on rail: $V_{rail\_wind\_u} := \gamma_{WS} \cdot w_{rail\_wind} \cdot \dfrac{L_{spc}}{2}$ $\quad V_{rail\_wind\_u} = 0.083\ kip$

$V_{rail\_u} = 0.43\ kip$ <- LL controls

Base Plate Design - Line Post w/ Axial Compression

Given: Plans

Cap width: $W_{cap} := 15.63\ in$

Distance from post to end of cap: $L_{end} := 8\ in$

Plate thickness: $t_p := .5\ in$

Plate length (perpendicular to fence): $N_{plate} := 8\ in$

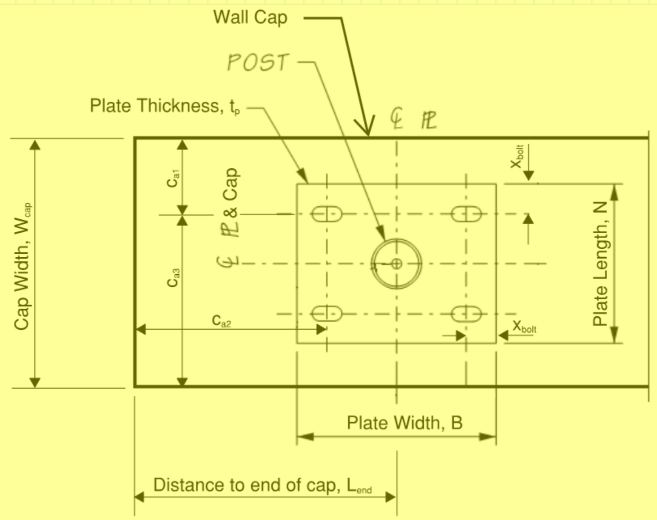
Plate width (parallel to fence): $B_{plate} := 10\ in$

Compressive Strength of Concrete: $f'_c := 4\ ksi$

Side clearance to anchor bolts: $x_{bolt} := 1.5\ in$

Base plate steel yield strength: $F_{y\_plate} := 36\ ksi$

Number of rails: $n_{rail} := 4$



Output:

Plate Area: $A_{plate} := N_{plate} \cdot B_{plate}$ $A_{plate} = 80\ in^2$

Distance from bolt to near face of cap: $c_{a1} := \frac{1}{2}\left(W_{cap} - N_{plate}\right) + x_{bolt}$ $c_{a1} = 5.315\ in$

Distance from outside bolt to end of cap: $c_{a2} := L_{end} - \frac{B_{plate}}{2} + x_{bolt}$ $c_{a2} = 4.5\ in$

Distance from bolt to far face of cap: $c_{a3} := W_{cap} - c_{a1}$ $c_{a3} = 10.315\ in$

Bearing Area taken to Be Same as Plate Area: $A_{bearing} := A_{plate}$ $A_{bearing} = 80\ in^2$ *Conservatively setting bearing area to the same as the plate.*

Max allowed bearing pressure: $f_{pu\_max} := \phi_{brg} \cdot min\left(0.85 \cdot f'_c \cdot \sqrt{\frac{A_{bearing}}{A_{plate}}}, 1.7 \cdot f'_c\right)$ $f_{pu\_max} = 2.04\ ksi$ *ACI Tbl. 14.5.6.1*

Max allowed bearing pressure line: $q_{max} := f_{pu\_max} \cdot B_{plate}$ $q_{max} = \left(2.448 \cdot 10^5\right) \frac{lbf}{ft}$

Post dead load on plate $P_{post\_DL} := w_{post} \cdot H_{post}$ $P_{post\_DL} = 0.026\ kip$

Rail dead load on plat: $P_{rail\_DL} := n_{rail} \cdot 2 \cdot V_{rail\_DL}$ $P_{rail\_DL} = 0.051\ kip$

Factored vertical load on plate: $P_u := \gamma_{DL} \cdot \left(P_{post\_DL} + P_{rail\_DL}\right)$ $P_u = 0.095\ kip$

Minimum length of area of bearing: $Y_{min} := \frac{P_u}{q_{max}}$ $Y_{min} = 0.005\ in$ *AISC DG#1 Eqn. 3.3.3*

Critical eccentricity distance: $e_{crit} := \frac{N_{plate}}{2} - \frac{Y_{min}}{2}$ $e_{crit} = 3.998\ in$ *AISC DG#1 Eqn. 3.3.7*

Eccentricity of loading: $e_{loading} := \frac{M_{post\_u}}{P_u}$ $e_{loading} = 441.7\ in$ *AISC DG#1 Eqn. 3.3.6*

Small moment check: $Small\_Moment\_Check :=$ if $e_{loading} \le e_{crit}$
  "Moment is small, no need for anchor bolts."
else
  "Moment is large, need anchor bolts."

$Small\_Moment\_Check =$ "Moment is large, need anchor bolts."

Distance from bolt to center of post: $f_{dim} := \frac{N_{plate}}{2} - x_{bolt}$ $f_{dim} = 2.5\ in$

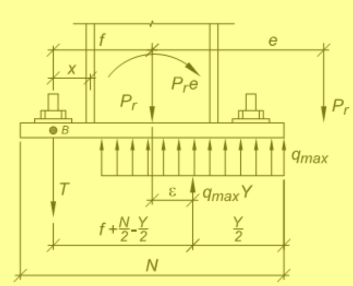


Fig. 3.4.1. Base plate with large moment.

Plate dimension check

$$Plate\_Dim\_Check := \text{if } \left(f_{dim} + \frac{N_{plate}}{2}\right)^2 \geq \frac{2 \cdot P_u \cdot \left(e_{loading} + f_{dim}\right)}{q_{max}}$$
$$\quad \text{“Plate dimensions are OK.”}$$
$$\text{else}$$
$$\quad \text{“Plate needs to be longer and/or wider.”}$$

$Plate\_Dim\_Check =$ "Plate dimensions are OK."

Length of bearing area centered at the eccentricity of this loading:

$$Y_{loading} := \left(f_{dim} + \frac{N_{plate}}{2}\right) - \sqrt{\left(f_{dim} + \frac{N_{plate}}{2}\right)^2 - \frac{2 \cdot P_u \cdot \left(e_{loading} + f_{dim}\right)}{q_{max}}}$$

$Y_{loading} = 0.327\ \boldsymbol{in}$

AISC DG#1 Eqn. 3.4.3

Required tensile resistance in anchor rods:

$$T_u := q_{max} \cdot Y_{loading} - P_u$$

$T_u = 6.571\ \boldsymbol{kip}$ *AISC DG#1 Eqn. 3.4.2*

Find minimum required thickness for plate based on bending at bearing interface:

Find bearing bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$m_{plate} := \frac{N_{plate} - 0.8\ OD_{post}}{2}$$

$m_{plate} = 2.85\ \boldsymbol{in}$

Calculating minimum thickness based on bearing:

$$t_{p\_brng\_req} := \text{if } Y_{loading} \geq m_{plate}$$
$$\quad 1.5 \cdot m_{plate} \cdot \sqrt{\frac{f_{pu\_max}}{F_{y\_plate}}}$$
$$\text{else}$$
$$\quad 2.11 \cdot \sqrt{\frac{f_{pu\_max} \cdot Y_{loading} \cdot \left(m_{plate} - \frac{Y_{loading}}{2}\right)}{F_{y\_plate}}}$$

$t_{p\_brng\_req} = 0.471\ \boldsymbol{in}$

AISC DG#1 Eqns. 3.3.14a-2 & 3.3.15a-2

Find minimum required thickness for plate based on bending at tension interface:

Find tension bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$x_{ten} := f_{dim} - \frac{0.8 \cdot OD_{post}}{2}$$

$x_{ten} = 1.35\ \boldsymbol{in}$

Calculating minimum thickness based on tension:

$$t_{p\_ten\_req} := 2.11 \cdot \sqrt{\frac{T_u \cdot x_{ten}}{B_{plate} \cdot F_{y\_plate}}}$$

$t_{p\_ten\_req} = 0.331\ \boldsymbol{in}$ *AISC DG#1 Eqn. 3.4.7a*

Controlling minimum required base plate thickness:

$$t_{p\_req} := \max\left(t_{p\_brng\_req}, t_{p\_ten\_req}\right)$$

$t_{p\_req} = 0.471\ \boldsymbol{in}$

Check chosen plate thickness:

$$Plate\_Thick\_Check := \text{if } t_p \geq t_{p\_req}$$
$$\quad \text{“Chosen plate thickness is adequate.”}$$
$$\text{else}$$
$$\quad \text{“Need a thicker plate.”}$$

$Plate\_Thick\_Check =$ "Chosen plate thickness is adequate."

Pipe to Plate Fillet Weld Connection Design

Given:

Minimum Fillet Weld Size: $w_{min} := \frac{1}{8}\ in$ *Min fillet weld size based on AISC Table J2-4*

Chosen fillet weld size $w := \frac{5}{16}\ in$

Weld material: $F_{EXX} := 70\ ksi$

Output:

Welded Connection to Base Plate Design:

Gross Length of Weld is Post Perimeter: $L_g := \pi \cdot OD_{post}$ $L_g = 9.032\ in$

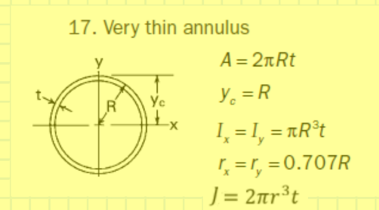
Effective Length of Weld: $L_w := L_g - 2 \cdot w$ $L_w = 8.407\ in$

Effective Throat Thickness: $t_e := min\left(w \cdot \sin(45\ deg), \frac{L_w}{4}\right)$ $t_e = 0.221\ in$ *AISC, Sect. J2, Pts. 2a & 2b*

Area of Weld: $A_w := L_w \cdot t_e$ $A_w = 1.858\ in^2$ *AISC, Sect. J2, Pts. 2a*

Moment of Inertia of circular fillet weld: $I_w := \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e$ $I_w = 2.062\ in^4$

Polar moment of Inertia of circular fillet weld: $J_w := 2\ \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e$ $J_w = 4.124\ in^4$



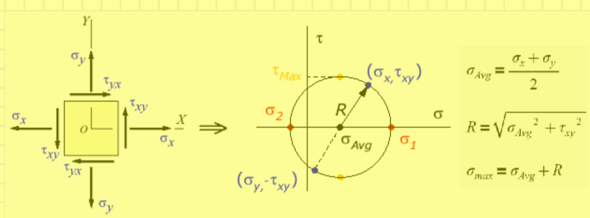
Determine design strength of weld:

Nominal strength of weld metal: $F_w := \phi_{fw} \cdot 0.6 \cdot F_{EXX}$ $F_w = 31.5\ ksi$ *AISC, Tbl. J2.5*

Normal stress caused by bending moment: $\sigma_b := \frac{M_{post\_u} \cdot \left(\frac{OD_{post}}{2}\right)}{I_w}$ $\sigma_b = 29.278\ ksi$ $\sigma = \frac{M}{S} = \frac{M \cdot c}{I}$

Stress caused by shearing force: $\tau_v := \frac{V_{post\_u}}{A_w}$ $\tau_v = 0.377\ ksi$

Resultant stress in weld from loading: $\sigma_{max} := \frac{\sigma_b}{2} + \sqrt{\left(\frac{\sigma_b}{2}\right)^2 + \tau_v^{\ 2}}$ $\sigma_{max} = 29.283\ ksi$

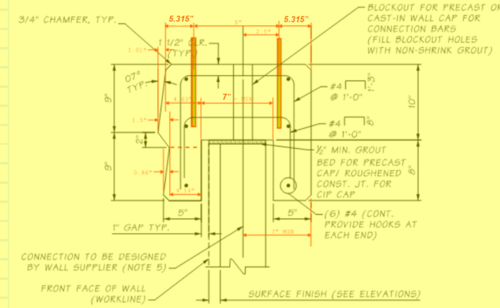


Check of weld thickness:

$Weld\_Design\_Check :=$ if $F_w \geq \sigma_{max}$
  "Chosen weld size is sufficient."
else
  "Need bigger fillet weld."

$Weld\_Design\_Check =$ "Chosen weld size is sufficient."

Anchor Bolt Connection Design



WALL CAP TYPICAL DETAIL

Given:

Number of anchor bolts resisting loads: $n_{ab} := 2$ *Only one side's bolts resist tension or shear.*

Bolts are specified as ASTM F1554 and Grade A36

Bolt diameter: $d_{ab} := \frac{5}{8}\ in$ *Plans*

Bolt area: $A_b := 0.307\ in^2$ *AISC Tbl. 7-18*

Bolt nominal yield stress strength: $F_{y\_bolt} := 36\ ksi$ *AISC Tbl. 2-3*

Bolt nominal ultimate tensile stress strength: $F_{u\_bolt} := 58\ ksi$

Bolt embedment: $h_{ef} := 5\ in$ *Plans*

Output:

Tension anchor bolt spacing: $s_1 := \frac{B_{plate} - 2 \cdot x_{bolt}}{n_{ab} - 1}$ $s_1 = 7\ in$

Bolt nominal tensile stress strength: $F_{nt} := 0.75 \cdot F_{u\_bolt}$ $F_{nt} = 43.5\ ksi$ *AISC Tbl. J3.2*

Bolt nominal shear stress strength: $F_{nv} := 0.40 \cdot F_{u\_bolt}$ $F_{nv} = 23.2\ ksi$ *AISC Tbl. J3.2, assuming threads within shear plane*

Ultimate tension load on one anchor bolt: $T_{u\_ab} := \frac{T_u}{n_{ab}}$ $T_{u\_ab} = 3.285\ kip$

Required shear stress on one bolt: $f_v := \frac{V_{post\_u}}{n_{ab} \cdot A_b}$ $f_v = 1.14\ ksi$

Bolt modified nominal tensile stress strength, modified for effects of shear stress: $F_{nt}' := min\left(F_{nt}, 1.3 \cdot F_{nt} - \frac{F_{nt}}{\phi_{ab} \cdot F_{nv}} \cdot f_v\right)$ $F_{nt}' = 43.5\ ksi$ *AISC Eqn. J3-3a*

Bolt factored tensile resistance: $\phi R_{n\_bolt} := \phi_{ab} \cdot F_{nt}' \cdot A_b$ $\phi R_{n\_bolt} = 10.016\ kip$ *AISC Eqn. J3-2*

Check of bolt tensile stress:

$$Bolt\_Tensile\_Check := \text{if } \phi R_{n\_bolt} \ge T_{u\_ab} \;\; \| \text{“Bolt is satisfactory.”} \;\; \text{else} \;\; \| \text{“Bolt is no good.”}$$

$Bolt\_Tensile\_Check =$ “Bolt is satisfactory.”

Continuing Anchor Bolt Connection Design per ACI 318

Outside diameter of anchor: $d_a := d_{ab}$ $d_a = 0.625\ in$

Critical edge distance for adhesive anchors: $c_{ac} := 2\ h_{ef}$ $c_{ac} = 10\ in$ *ACI 17.7.6*

Steel strength of anchor in tension (ACI 17.4.1)

Steel tension strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in tension (ACI 17.4.2)

Check bolt group action for tension concrete breakout: *ACI 17.2.1.1*

$$Group\_Tension\_Breakout\_Check := \text{if } s_1 \le 3 \cdot h_{ef} \;\; \| \text{“Bolts act in group.”} \;\; \text{else} \;\; \| \text{“Bolts act singly.”}$$

$Group\_Tension\_Breakout\_Check =$ “Bolts act in group.”

Theoretical projected influence area of a single bolt far from an edge: $A_{Nco} := 9 \cdot h_{ef}^2$ $A_{Nco} = 225\ in^2$ *ACI Eqn. 17.4.2.1c*

Actual projected influence area for bolt(s): $A_{Nc} := min\left(\left(c_{a1} + 1.5 \cdot h_{ef}\right) \cdot \left(1.5 \cdot h_{ef} + min\left(s_1, 3 \cdot h_{ef}\right) + min\left(1.5 \cdot h_{ef}, c_{a2}\right)\right), n_{ab} \cdot A_{Nco}\right)$ $A_{Nc} = 243.485\ in^2$ *ACI Fig. R17.4.2.1*

Concrete tension breakout strength coefficient: $k_c := 17$ *Value of 17 for post-installed anchors, per ACI 17.4.2.2*

Basic concrete tension breakout strength for single anchor: $N_b := k_c \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lbf$ $N_b = 12.021\ kip$ *ACI Eqn. 17.4.2.2a*

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_N} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.4.2.4*

Factor for anchor bolts near an edge: $\Psi_{ed\_N} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a1}}{1.5 \cdot h_{ef}}\right)$ $\Psi_{ed\_N} = 0.913$ *ACI Eqn. 17.4.2.5b*

Factor for anchor bolts in un-cracked concrete: $\Psi_{c\_N} := 1.4$ *Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.* *ACI 17.4.2.6*

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement: $\Psi_{cp\_N} := min\left(1.0, \max\left(\frac{c_{a1}}{c_{ac}}, \frac{1.5 \cdot h_{ef}}{c_{ac}}\right)\right)$ $\Psi_{cp\_N} = 0.75$ *ACI Eqn. 17.4.2.7b*

Nominal concrete tension breakout strength: $\phi N_{cbg} := \phi_{adh} \cdot \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ec\_N} \cdot \Psi_{ed\_N} \cdot \Psi_{c\_N} \cdot \Psi_{cp\_N} \cdot N_b$ $\phi N_{cbg} = 8.102\ kip$ *ACI Eqn. 17.4.2.1b*

Check of concrete tension breakout failure:

```
Concrete_Tension_Breakout_Check := if ϕNcbg ≥ nab • Tu_ab
                                       “Bolt is satisfactory.”
                                   else
                                       “Bolt is no good.”
```

Concrete_Tension_Breakout_Check = “Bolt is satisfactory.”

Pullout strength cast-in, post-installed expansion, or undercut anchor in tension (ACI 17.4.3)

Proposed anchors are post-installed adhesive, not headed studs or bolts, expansion anchors, or undercut anchors; so, no check is required.

Concrete side-face blowout strength of headed anchor in tension (ACI 17.4.4)

Proposed anchors are post-installed adhesive, not headed studs or bolts; so, no check is required.

Bond strength of adhesive anchor in tension (ACI 17.4.5)

Minimum bond stress for HY 200 Epoxy per HILTI ESR-3187:

$$\tau_{uncr\_HY\_200} := 0.65 \cdot \left(\frac{f'_c}{2500\ psi}\right)^{0.1} \cdot 2220\ psi = 1512.441\ psi$$

Per HILTI ESR-3187 Table 14, basic un-cracked bond strength is 2,220 psi; this value is factored by a straight 0.65 for either wet or dry installation conditions and by a small boost from concrete strength higher than 2,500 psi

Minimum bond stress for HIT-RE 500 Epoxy per HILTI ESR-3814:

$$\tau_{uncr\_HIT\_RE\_500} := 0.65 \cdot \left(\frac{f'_c}{2500\ psi}\right)^{0.15} \cdot 2210\ psi = 1541.429\ psi$$

Per HILTI ESR-3814 Table 12, basic un-cracked bond strength is 2,210 psi. This value is based on diamond coring and roughening afterwards; it is lower than being hammer-drilled with carbide bit. The socket must be roughened if coring with a diamond bit; this should be written on the plans. Factors are a straight 0.65 reduction factor independent of wet or dry concrete conditions during installation and a small boost for using concrete higher than 2,500 psi. The smaller factor for cracked concrete is used since no supplementary rebar is being provided; this also matches with reduction factor below.

Minimum bond stress strength: $\tau_{uncr} := min\left(\tau_{uncr\_HY\_200}, \tau_{uncr\_HIT\_RE\_500}\right)$ $\tau_{uncr} = 1512.441\ psi$

Distance to edge of project influence area: $c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100\ psi}}$ $c_{Na} = 7.329\ in$ *ACI Eqn. 17.4.5.1d*

Check if anchor bolts act in group for bond failure: *ACI 17.2.1.1*

```
Group_Bond_Failure_Check := if s1 ≤ 2 cNa
                                “Bolts act in group.”
                            else
                                “Bolts act singly.”
```

Group_Bond_Failure_Check = “Bolts act in group.”

Theoretical projected influence area of a single bolt far from an edge: $A_{Nao} := (2\ c_{Na})^2$ $A_{Nao} = 214.835\ in^2$ ACI Eqn. 17.4.5.1c

Actual projected influence area for bolt(s): $A_{Na} := min\left(\left(c_{Na} + min\left(s_1, 2 \cdot c_{Na}\right) + min\left(c_{Na}, c_{a2}\right)\right) \cdot \left(c_{a1} + c_{Na}\right), n_{ab} \cdot A_{Nao}\right)$ $A_{Na} = 238.062\ in^2$ ACI Fig. R17.4.5.1

Basic bond strength of adhesive anchor: $N_{ba} := \tau_{uncr} \cdot \pi \cdot d_a \cdot h_{ef}$ $N_{ba} = 14.848\ kip$ ACI Eqn. 17.4.5.2

Concrete is not light weight; so, lambda-a is set to 1.0; per ACI 17.4.5.2, un-cracked bond stress may be used.

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_Na} := 1.0$ *Anchor bolts are not loaded eccentrically.* ACI 17.4.5.3

Factor for anchor bolts near an edge: $\Psi_{ed\_Na} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a1}}{c_{Na}}\right)$ $\Psi_{ed\_Na} = 0.918$ ACI Eqn. 17.4.5.4b

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement: $\Psi_{cp\_Na} := min\left(1.0, \max\left(\frac{c_{a1}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right)\right)$ $\Psi_{cp\_Na} = 0.733$ ACI Eqn. 17.4.5.5b

Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

Nominal bond strength of the adhesive anchor(s): $\phi N_{ag} := \phi_{adh} \cdot \frac{A_{Na}}{A_{Nao}} \cdot \Psi_{ec\_Na} \cdot \Psi_{ed\_Na} \cdot \Psi_{cp\_Na} \cdot N_{ba}$ $\phi N_{ag} = 7.192\ kip$ ACI Eqn. 17.4.5.1b

Check of bolt bond stress failure:

$Bond\_Stress\_Check :=$ if $\phi N_{ag} \ge n_{ab} \cdot T_{u\_ab}$
  "Bolt is satisfactory."
else
  "Bolt is no good."

$Bond\_Stress\_Check =$ "Bolt is satisfactory."

Steel strength of anchor in shear (17.5.1)

Steel shear strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in shear (17.5.2)

Check bolt group action for shear concrete breakout: ACI 17.2.1.1

$Group\_Shear\_Breakout\_Check :=$ if $s_1 \le 3 \cdot c_{a1}$
  "Bolts act in group."
else
  "Bolts act singly."

$Group\_Shear\_Breakout\_Check =$ "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Vco} := 4.5 \cdot {c_{a1}}^2$ $A_{Vco} = 127.122\ in^2$ ACI Eqn. 17.5.2.1c

Actual projected influence area for bolt(s): $A_{Vc} := min\left(1.5 \cdot c_{a1} \cdot \left(1.5 \cdot c_{a1} + min\left(s_1, 3 \cdot c_{a1}\right) + min\left(1.5 \cdot c_{a1}, c_{a2}\right)\right), n_{ab} \cdot A_{Vco}\right)$ $A_{Vc} = 155.245\ in^2$ ACI Fig. R17.5.2.1b

Load bearing length: $l_e := h_{ef}$ $l_e = 5\ in$ ACI 17.5.2.2

Basic concrete breakout strength in shear for single anchor:

$$V_b := min\left(\left(7\left(\frac{l_e}{d_a}\right)^{0.2} \cdot \sqrt{\frac{d_a}{in}}\right) \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5}, 9 \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5}\right) \cdot lbf \quad V_b = 6.5\ kip$$

Concrete is not light weight; so, lambda-a is set to 1.0. ACI Eqns. 17.5.2.2a & 17.5.2.2b

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_V} := 1.0$ *Anchor bolts are not loaded eccentrically.* ACI 17.5.2.5

Factor for anchor bolts near an edge: $\Psi_{ed\_V} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}\right)$ $\Psi_{ed\_V} = 0.869$ ACI Eqns. 17.5.2.6a & 17.5.2.6b

Factor for anchor bolts in un-cracked concrete: $\Psi_{c\_V} := 1.4$ *Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.* ACI 17.5.2.7

Factor for small embedment $\Psi_{h\_V} := min\left(1.0, \sqrt{\frac{1.5 \cdot c_{a1}}{h_{ef}}}\right)$ $\Psi_{h\_V} = 1$ *ACI Eqn. 17.5.2.8*

Nominal concrete shear breakout strength: $\phi V_{cbg} := \phi_{adh} \cdot \frac{A_{Vc}}{A_{Vco}} \cdot \Psi_{ec\_V} \cdot \Psi_{ed\_V} \cdot \Psi_{c\_V} \cdot \Psi_{h\_V} \cdot V_b$ $\phi V_{cbg} = 6.28\ kip$ *ACI Eqn. 17.5.2.1b*

Check of concrete shear breakout failure: Concrete_Shear_Breakout_Check := if $\phi V_{cbg} \geq V_{post\_u}$
  "Bolt is satisfactory."
else
  "Bolt is no good."

Concrete_Shear_Breakout_Check = "Bolt is satisfactory."

Concrete pryout strength of anchor in shear (17.5.3)

Basic concrete pryout strength of a single anchor in shear: $\phi N_{cpg} := min\left(\phi N_{ag}, \phi N_{cbg}\right)$ $\phi N_{cpg} = 7.192\ kip$ *ACI 17.5.3.1*

Concrete pryout strength in shear coefficient: $k_{cp} :=$ if $h_{ef} < 2.5\ in$
  1.0
else
  2.0

$k_{cp} = 2$ *ACI 17.5.3.1*

Nominal concrete pryout strength of anchor(s) in shear: $\phi V_{cpg} := k_{cp} \cdot \phi N_{cpg}$ $\phi V_{cpg} = 14.384\ kip$ *ACI Eqn. 17.5.3.1b*

Check of concrete pryout strength in shear: Concrete_Shear_Pryout_Check := if $\phi V_{cpg} \geq V_{post\_u}$
  "Bolt is satisfactory."
else
  "Bolt is no good."

Concrete_Shear_Pryout_Check = "Bolt is satisfactory."

Vertical Interior Post and Horizontal Rail Design

Given:

Post Height: $H_{post} := 42\ in$ Plans

Post and Rail Yield Strength: $F_y := 50\ ksi$ ASTM F1043

Post and Rail Modulus of Elasticity: $E_s := 29000\ ksi$

Post and Rail Ultimate Strength: $F_u := 58\ ksi$

Properties for ASTM F1043 IC 1-7/8" Pipe for Interior Posts

Post OD: $OD_{post} := 1.900\ in$

Post Thickness: $t_{post} := 0.12\ in$

Properties for ASTM F1043 IC 1-5/8" Pipe for Rails

Rail OD: $OD_{rail} := 1.66\ in$

Rail Thickness: $t_{rail} := 0.111\ in$

Design Point Live Load $P_{LL} := 200\ lbf$ AASHTO 13.8.2

Design Uniform Live Load $w_{LL} := 0\ plf$ AASHTO 13.8.2

Post spacing: $L_{spc} := 8\ ft$ Plans

Weight of chain link fence: $f_{clf} := 0.48\ psf$

Design wind load from chain link fence: $f_{wind} := 15\ psf$ AASHTO 13.8.2

Load Factors (AASHTO Tbl. 3.4.1-1):

PL Load Factor: $\gamma_{PL} := 1.75$

DC Load Factor: $\gamma_{DL} := 1.25$

WS Load Factor: $\gamma_{WS} := 1.00$

Resistance Factors:

Steel Flexure (AASHTO 6.5.4.2): $\phi_f := 1.00$

Steel Shear (AASHTO 6.5.4.2): $\phi_v := 1.00$

Tension, Yielding in Gross Section: $\phi_y := 0.95$

Bending (AISC F1): $\phi_b := 0.90$

Shear (AISC G1): $\phi_{v\_AISC} := 0.90$

Bearing (AISC DG#1): $\phi_{brg} := 0.60$

Fillet Weld (AISC Tbl. J2.5): $\phi_{fw} := 0.75$

Bolts (AISC J3.6 & J3.7): $\phi_{ab} := 0.75$

Adhesive Anchor Bolts (ACI 17.3.3, Condition B, Category 1): $\phi_{adh} := 0.65$

Steel weight density: $\gamma_{steel} := 490\ pcf$

ASTM F1043 Group IC Electric Resistant Welded 50,000 psi yield steel pipe

Trade Reference	Decimal O.D. Equivalent		Pipe wall Thickness		Weight		Section Modulus		x	Min. Yield Strength		=	Max Bending Moment	Calculated Load (lbs)		
														10' Free	Cantilever	
O.D.	inches	(mm)	inches	(mm)	lb./ft.	(kg/m)	inches³	(mm³)	x	psi	(Mpa)	=	Lb.In.	Supported	4'	6'
1 5/8"	1.660	42.16	0.111	2.82	1.84	2.74	0.1962	4.98	x	50000	345	=	9810	327	204	136
1 7/8"	1.900	48.26	0.120	3.05	2.28	3.39	0.2810	7.14	x	50000	345	=	14050	468	293	195
2 3/8"	2.375	60.33	0.130	3.30	3.12	4.64	0.4881	12.40	x	50000	345	=	24405	814	508	339
2 7/8"	2.875	73.03	0.160	4.06	4.64	6.91	0.8778	22.30	x	50000	345	=	43890	1463	914	610
3 1/2"	3.500	88.90	0.160	4.06	5.71	8.50	1.3408	34.06	x	50000	345	=	67042	2235	1397	931
4"	4.000	101.60	0.160	4.06	6.57	9.78	1.7820	45.26	x	50000	345	=	89098	2970	1856	1237
4 1/2"	4.500	114.30	0.160	4.14	7.42	11.04	2.2859	57.99	X	50000	345	=	114295	3810	5486	1587

Output:

Post Section Properties:

Post inside diameter: $ID_{post} := OD_{post} - 2 \cdot t_{post}$ $ID_{post} = 1.66\ in$

Post Area: $A_{post} := 0.785398 \cdot \left(OD_{post}^{\ 2} - ID_{post}^{\ 2}\right)$ $A_{post} = 0.671\ in^2$

Post Unit Weight: $w_{post} := \gamma_{steel} \cdot A_{post}$ $w_{post} = 2.283\ plf$

Post centroid: $c_{post} := 0.5 \cdot OD_{post}$ $c_{post} = 0.95\ in$

Post Moment of Inertial: $I_{post} := 0.049087 \cdot \left(OD_{post}^{\ 4} - ID_{post}^{\ 4}\right)$ $I_{post} = 0.267\ in^4$

Post Section Modulus: $S_{post} := \dfrac{I_{post}}{c_{post}}$ $S_{post} = 0.281\ in^3$

Post Plastic Section Modulus: $Z_{post} := \dfrac{OD_{post}^{\ 3} - ID_{post}^{\ 3}}{6}$ $Z_{post} = 0.381\ in^3$

Rail Section Properties:

Rail inside diameter: $ID_{rail} := OD_{rail} - 2 \cdot t_{rail}$ $ID_{rail} = 1.438\ in$

Rail Area: $A_{rail} := 0.785398 \cdot \left(OD_{rail}^{\ 2} - ID_{rail}^{\ 2}\right)$ $A_{rail} = 0.54\ in^2$

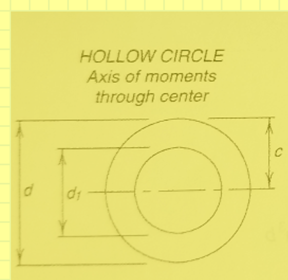
Rail Unit Weight: $w_{rail} := \gamma_{steel} \cdot A_{rail}$ $w_{rail} = 1.838\ plf$

Rail centroid: $c_{rail} := 0.5 \cdot OD_{rail}$ $c_{rail} = 0.83\ in$

Rail Moment of Inertial: $I_{rail} := 0.049087 \cdot \left(OD_{rail}^{\ 4} - ID_{rail}^{\ 4}\right)$ $I_{rail} = 0.163\ in^4$

Rail Section Modulus: $S_{rail} := \dfrac{I_{rail}}{c_{rail}}$ $S_{rail} = 0.196\ in^3$

Rail Plastic Section Modulus: $Z_{rail} := \dfrac{OD_{rail}^{\ 3} - ID_{rail}^{\ 3}}{6}$ $Z_{rail} = 0.267\ in^3$



$$A = \frac{\pi(d^2 - d_1^2)}{4} = .785398\,(d^2 - d_1^2)$$

$$c = \frac{d}{2}$$

$$I = \frac{\pi(d^4 - d_1^4)}{64} = .049087\,(d^4 - d_1^4)$$

$$S = \frac{\pi(d^4 - d_1^4)}{32d} = .098175\ \frac{d^4 - d_1^4}{d}$$

$$r = \frac{\sqrt{d^2 + d_1^2}}{4}$$

$$Z = \frac{d^3}{6} - \frac{d_1^3}{6}$$

Post concentrated live load applied at top rail: $P_{post\_LL} := P_{LL} + w_{LL} \cdot L_{spc} = 0.2\ kip$ $\quad P_{post\_LL} = 0.2\ kip$ *AASHTO Eqn. 13.8.2-1*

Post moment loading from live load: $M_{post\_LL} := P_{post\_LL} \cdot H_{post} = 8400\ lbf \cdot in$ $\quad M_{post\_LL} = 8400\ lbf \cdot in$ *Post treated as cantilevered beam*

Post shear from live load: $V_{post\_LL} := P_{post\_LL}$ $\quad V_{post\_LL} = 0.2\ kip$

Rail moment from live load applied: $M_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}^2}{8} + \frac{P_{LL} \cdot L_{spc}}{4}$ $\quad M_{rail\_LL} = 4800\ lbf \cdot in$

Rail treated as simply supported beam with vertical and horizontal live loads combined into resultant direction.

Rail moment from dead load: $M_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}^2}{8} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}^2}{8}$ $\quad M_{rail\_DL} = 257.093\ lbf \cdot in$

Rail shear from live load: $V_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}}{2} + \frac{P_{LL}}{2}$ $\quad V_{rail\_LL} = 0.1\ kip$

Rail shear from dead load: $V_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}}{2} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}}{2}$ $\quad V_{rail\_DL} = 0.011\ kip$

Factored Shear Load on Post: $V_{post\_u} := \gamma_{PL} \cdot V_{post\_LL}$ $\quad V_{post\_u} = 0.35\ kip$

Factored Moment Load on Post: $M_{post\_u} := \gamma_{PL} \cdot M_{post\_LL}$ $\quad M_{post\_u} = 14700\ lbf \cdot in$

AASHTO load factors used instead of ASCE load factors found in AISC and ACI. This is acceptable as it is more conservative.

Factored Shear Load on Rail: $V_{rail\_u} := \gamma_{PL} \cdot V_{rail\_LL} + \gamma_{DL} \cdot V_{rail\_DL}$ $\quad V_{rail\_u} = 0.188\ kip$

Factored Moment Load on Rail: $M_{rail\_u} := \gamma_{PL} \cdot M_{rail\_LL} + \gamma_{DL} \cdot M_{rail\_DL}$ $\quad M_{rail\_u} = 8721.366\ lbf \cdot in$

Vertical dead load was combined directly with live load resultant since it was so small compared to the live load.

Post Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area: $A_g := A_{post}$ $\quad A_g = 0.671\ in^2$

Distance from Max to 0 Shear: $L_v := H_{post}$ $\quad L_v = 42\ in$

Critical Strength for Shear:

$$F_{cr} := min\left(0.58 \cdot F_y, \max\left(\frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{post}}} \left(\frac{OD_{post}}{t_{post}}\right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left(\frac{OD_{post}}{t_{post}}\right)^{\frac{3}{2}}}\right)\right)$$

$F_{cr} = 29\ ksi$ *AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3*

Factored nominal shear resistance: $\phi V_n := \phi_v \cdot 0.5\ F_{cr} \cdot A_g$ $\quad \phi V_n = 9.73\ kip$ *AASHTO Eqn. 6.12.1.2.3c-1*

Post Shear Check: $\frac{\phi V_n}{V_{post\_u}} = 27.8$

$Post\_Shear\_Check :=$ if $\frac{\phi V_n}{V_{post\_u}} \geq 1.0$
"Post shear strength is satisfactory."
else
"Post is not satisfactory."

$Post\_Shear\_Check =$ "Post shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section: $Check\_Compact :=$ if $\frac{OD_{post}}{t_{post}} \leq \frac{0.07 \cdot E_s}{F_y}$
"Section is compact. Local buckling does not apply."
else
"Section is not compact. Check wall slenderness."

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

$Check\_Compact =$ "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance: $\phi M_n := \phi_f \cdot F_y \cdot Z_{post}$ $\quad \phi M_n = 19.039\ kip \cdot in$ *AASHTO Eqn. 6.12.2.2.3-1*

Post Flexural Check: $\frac{\phi M_n}{M_{post\_u}} = 1.295$

$Post\_Flex\_Check := \text{if } \frac{\phi M_n}{M_{post\_u}} \geq 1.0$
  "Post flexural strength is satisfactory."
else
  "Post is not satisfactory."

$Post\_Flex\_Check =$ "Post flexural strength is satisfactory."

Rail Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area: $A_g := A_{rail}$ $A_g = 0.54\ in^2$

Distance from Max to 0 Shear: $L_v := \frac{L_{spc}}{2}$ $L_v = 48\ in$

Critical Strength for Shear:

$$F_{cr} := min\left(0.58 \cdot F_y, \max\left(\frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{rail}}}\left(\frac{OD_{rail}}{t_{rail}}\right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left(\frac{OD_{rail}}{t_{rail}}\right)^{\frac{3}{2}}}\right)\right)$$

$F_{cr} = 29\ ksi$ *AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3*

Factored Nominal Shear Resistance: $\phi V_n := \phi_v \cdot 0.5\ F_{cr} \cdot A_g$ $\phi V_n = 7.832\ kip$ *AASHTO Eqn. 6.12.1.2.3c-1*

Rail Shear Check: $\frac{\phi V_n}{V_{rail\_u}} = 41.575$

$Rail\_Shear\_Check := \text{if } \frac{\phi V_n}{V_{rail\_u}} \geq 1.0$
  "Rail shear strength is satisfactory."
else
  "Rail is not satisfactory."

$Rail\_Shear\_Check =$ "Rail shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

$Check\_Compact := \text{if } \frac{OD_{rail}}{t_{rail}} \leq \frac{0.07 \cdot E_s}{F_y}$
  "Section is compact. Local buckling does not apply."
else
  "Section is not compact. Check wall slenderness."

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

$Check\_Compact =$ "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance: $\phi M_n := \phi_f \cdot F_y \cdot Z_{rail}$ $\phi M_n = 13.339\ kip \cdot in$ *AASHTO Eqn. 6.12.2.2.3-1*

Post Flexural Check: $\frac{\phi M_n}{M_{rail\_u}} = 1.53$

$Rail\_Flex\_Check := \text{if } \frac{\phi M_n}{M_{rail\_u}} \geq 1.0$
  "Rail flexural strength is satisfactory."
else
  "Rail is not satisfactory."

$Rail\_Flex\_Check =$ "Rail flexural strength is satisfactory."

Confirming that Wind Loading Doesn't Control:

Per last paragraph of AASHTO 13.8.2, the wind load on the chain link fence is not applied simultaneously with the live load.

Uniform wind load on post: $w_{post\_wind} := f_{wind} \cdot L_{spc}$ $w_{post\_wind} = 120\ plf$

Design moment from wind on post: $M_{post\_wind\_u} := \gamma_{WS} \cdot \dfrac{w_{post\_wind} \cdot H_{post}^{2}}{2}$ $M_{post\_wind\_u} = 8820\ lbf \cdot in$

$M_{post\_u} = 14700\ lbf \cdot in$ <- LL controls

Design shear from wind on post: $V_{post\_wind\_u} := \gamma_{WS} \cdot w_{post\_wind} \cdot H_{post}$ $V_{post\_wind\_u} = 0.42\ kip$

$V_{post\_u} = 0.35\ kip$ <- LL controls

Uniform wind on rail: $w_{rail\_wind} := f_{wind} \cdot \dfrac{H_{post}}{2}$ $w_{rail\_wind} = 26.25\ plf$

Design moment from wind on rail: $M_{rail\_wind\_u} := \gamma_{WS} \cdot \dfrac{w_{rail\_wind} \cdot L_{spc}^{2}}{8}$ $M_{rail\_wind\_u} = 2520\ lbf \cdot in$

$M_{rail\_u} = 8721.366\ lbf \cdot in$ <- LL controls

Design shear from wind on rail: $V_{rail\_wind\_u} := \gamma_{WS} \cdot w_{rail\_wind} \cdot \dfrac{L_{spc}}{2}$ $V_{rail\_wind\_u} = 0.105\ kip$

$V_{rail\_u} = 0.188\ kip$ <- LL controls

Base Plate Design - Line Post w/ Axial Compression

Given: *Plans*

Cap width: $W_{cap} := 15.63\ in$

Distance from post to end of cap: $L_{end} := 96\ in$

Plate thickness: $t_p := 0.5\ in$

Plate length (perpendicular to fence): $N_{plate} := 8\ in$

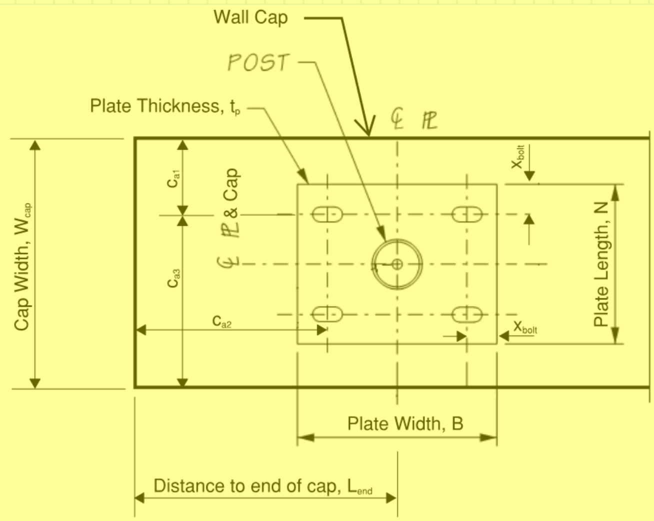
Plate width (parallel to fence): $B_{plate} := 10\ in$

Compressive Strength of Concrete: $f'_c := 4\ ksi$

Side clearance to anchor bolts: $x_{bolt} := 1.5\ in$

Base plate steel yield strength: $F_{y\_plate} := 36\ ksi$

Number of rails: $n_{rail} := 2$



Output:

Plate Area: $A_{plate} := N_{plate} \cdot B_{plate}$ $\quad A_{plate} = 80\ in^2$

Distance from bolt to near face of cap: $c_{a1} := \frac{1}{2}\left(W_{cap} - N_{plate}\right) + x_{bolt}$ $\quad c_{a1} = 5.315\ in$

Distance from outside bolt to end of cap: $c_{a2} := L_{end} - \frac{B_{plate}}{2} + x_{bolt}$ $\quad c_{a2} = 92.5\ in$

Distance from bolt to far face of cap: $c_{a3} := W_{cap} - c_{a1}$ $\quad c_{a3} = 10.315\ in$

Bearing Area taken to Be Same as Plate Area: $A_{bearing} := A_{plate}$ $\quad A_{bearing} = 80\ in^2$ *Conservatively setting bearing area to the same as the plate.*

Max allowed bearing pressure: $f_{pu\_max} := \phi_{brg} \cdot min\left(0.85 \cdot f'_c \cdot \sqrt{\frac{A_{bearing}}{A_{plate}}}, 1.7 \cdot f'_c\right)$ $\quad f_{pu\_max} = 2.04\ ksi$ *ACI Tbl. 14.5.6.1*

Max allowed bearing pressure line: $q_{max} := f_{pu\_max} \cdot B_{plate}$ $\quad q_{max} = \left(2.448 \cdot 10^5\right) \frac{lbf}{ft}$

Post dead load on plate $P_{post\_DL} := w_{post} \cdot H_{post}$ $\quad P_{post\_DL} = 0.008\ kip$

Rail dead load on plat: $P_{rail\_DL} := n_{rail} \cdot 2 \cdot V_{rail\_DL}$ $\quad P_{rail\_DL} = 0.043\ kip$

Factored vertical load on plate: $P_u := \gamma_{DL} \cdot \left(P_{post\_DL} + P_{rail\_DL}\right)$ $\quad P_u = 0.064\ kip$

Minimum length of area of bearing: $Y_{min} := \frac{P_u}{q_{max}}$ $\quad Y_{min} = 0.003\ in$ *AISC DG#1 Eqn. 3.3.3*

Critical eccentricity distance: $e_{crit} := \frac{N_{plate}}{2} - \frac{Y_{min}}{2}$ $\quad e_{crit} = 3.998\ in$ *AISC DG#1 Eqn. 3.3.7*

Eccentricity of loading: $e_{loading} := \frac{M_{post\_u}}{P_u}$ $\quad e_{loading} = 231.31\ in$ *AISC DG#1 Eqn. 3.3.6*

Small moment check: $Small\_Moment\_Check :=$ if $e_{loading} \le e_{crit}$
  "Moment is small, no need for anchor bolts."
else
  "Moment is large, need anchor bolts."

$Small\_Moment\_Check =$ "Moment is large, need anchor bolts."

Distance from bolt to center of post: $f_{dim} := \frac{N_{plate}}{2} - x_{bolt}$ $\quad f_{dim} = 2.5\ in$

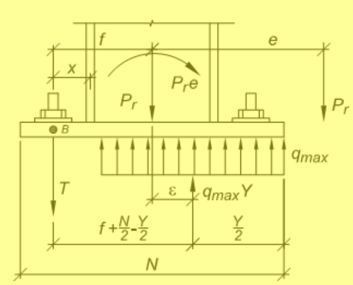


Fig. 3.4.1. Base plate with large moment.

Plate dimension check

$$Plate\_Dim\_Check := \text{if } \left(f_{dim} + \frac{N_{plate}}{2}\right)^2 \ge \frac{2 \cdot P_u \cdot \left(e_{loading} + f_{dim}\right)}{q_{max}}$$
"Plate dimensions are OK."
else
"Plate needs to be longer and/or wider."

$Plate\_Dim\_Check =$ "Plate dimensions are OK."

Length of bearing area centered at the eccentricity of this loading:

$$Y_{loading} := \left(f_{dim} + \frac{N_{plate}}{2}\right) - \sqrt{\left(f_{dim} + \frac{N_{plate}}{2}\right)^2 - \frac{2 \cdot P_u \cdot \left(e_{loading} + f_{dim}\right)}{q_{max}}}$$

$Y_{loading} = 0.113\ \textbf{in}$

AISC DG#1 Eqn. 3.4.3

Required tensile resistance in anchor rods: $T_u := q_{max} \cdot Y_{loading} - P_u$ $T_u = 2.242\ \textbf{kip}$ *AISC DG#1 Eqn. 3.4.2*

Find minimum required thickness for plate based on bending at bearing interface:

Find bearing bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$m_{plate} := \frac{N_{plate} - 0.8\ OD_{post}}{2}$$

$m_{plate} = 3.24\ \textbf{in}$

Calculating minimum thickness based on bearing:

$$t_{p\_brng\_req} := \text{if } Y_{loading} \ge m_{plate}$$
$$1.5 \cdot m_{plate} \cdot \sqrt{\frac{f_{pu\_max}}{F_{y\_plate}}}$$
else
$$2.11 \cdot \sqrt{\frac{f_{pu\_max} \cdot Y_{loading} \cdot \left(m_{plate} - \frac{Y_{loading}}{2}\right)}{F_{y\_plate}}}$$

$t_{p\_brng\_req} = 0.301\ \textbf{in}$

AISC DG#1 Eqns. 3.3.14a-2 & 3.3.15a-2

Find minimum required thickness for plate based on bending at tension interface:

Find tension bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$x_{ten} := f_{dim} - \frac{0.8 \cdot OD_{post}}{2}$$

$x_{ten} = 1.74\ \textbf{in}$

Calculating minimum thickness based on tension:

$$t_{p\_ten\_req} := 2.11 \cdot \sqrt{\frac{T_u \cdot x_{ten}}{B_{plate} \cdot F_{y\_plate}}}$$

$t_{p\_ten\_req} = 0.22\ \textbf{in}$ *AISC DG#1 Eqn. 3.4.7a*

Controlling minimum required base plate thickness: $t_{p\_req} := \max\left(t_{p\_brng\_req}, t_{p\_ten\_req}\right)$ $t_{p\_req} = 0.301\ \textbf{in}$

Check chosen plate thickness:

$$Plate\_Thick\_Check := \text{if } t_p \ge t_{p\_req}$$
"Chosen plate thickness is adequate."
else
"Need a thicker plate."

$Plate\_Thick\_Check =$ "Chosen plate thickness is adequate."

Pipe to Plate Fillet Weld Connection Design

Given:

Minimum Fillet Weld Size: $w_{min} := \frac{1}{8}\ in$ *Min fillet weld size based on AISC Table J2-4*

Chosen fillet weld size $w := \frac{5}{16}\ in$

Weld material: $F_{EXX} := 70\ ksi$

Output:

Welded Connection to Base Plate Design:

Gross Length of Weld is Post Perimeter: $L_g := \pi \cdot OD_{post}$ $L_g = 5.969\ in$

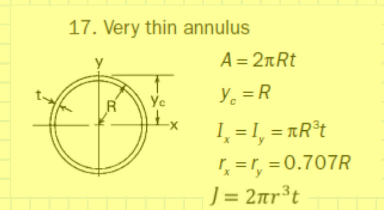
Effective Length of Weld: $L_w := L_g - 2 \cdot w$ $L_w = 5.344\ in$

Effective Throat Thickness: $t_e := min\left(w \cdot \sin(45\ deg), \frac{L_w}{4}\right)$ $t_e = 0.221\ in$ *AISC, Sect. J2, Pts. 2a & 2b*

Area of Weld: $A_w := L_w \cdot t_e$ $A_w = 1.181\ in^2$ *AISC, Sect. J2, Pts. 2a*

Moment of Inertia of circular fillet weld: $I_w := \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e$ $I_w = 0.595\ in^4$

Polar moment of Inertia of circular fillet weld: $J_w := 2\ \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e$ $J_w = 1.19\ in^4$



Determine design strength of weld:

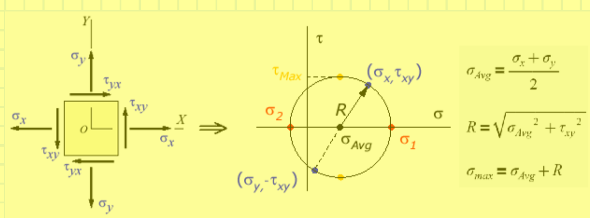
Nominal strength of weld metal: $F_w := \phi_{fw} \cdot 0.6 \cdot F_{EXX}$ $F_w = 31.5\ ksi$ *AISC, Tbl. J2.5*

Normal stress caused by bending moment: $\sigma_b := \frac{M_{post\_u} \cdot \left(\frac{OD_{post}}{2}\right)}{I_w}$ $\sigma_b = 23.463\ ksi$

$$\sigma = \frac{M}{S} = \frac{M \cdot c}{I}$$

Stress caused by shearing force: $\tau_v := \frac{V_{post\_u}}{A_w}$ $\tau_v = 0.296\ ksi$

Resultant stress in weld from loading: $\sigma_{max} := \frac{\sigma_b}{2} + \sqrt{\left(\frac{\sigma_b}{2}\right)^2 + \tau_v^{\ 2}}$ $\sigma_{max} = 23.467\ ksi$



Check of weld thickness:

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Weld_Design_Check := if F_w ≥ σ_max
                       ‖ "Chosen weld size is sufficient."
                     else
                       ‖ "Need bigger fillet weld."
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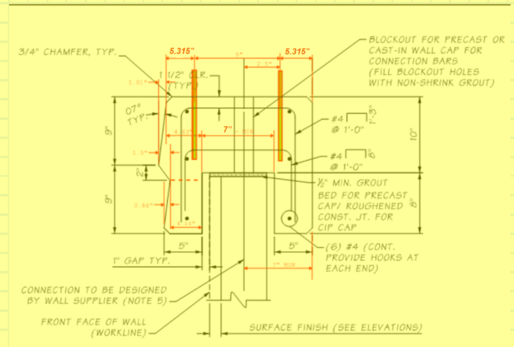
$Weld\_Design\_Check =$ "Chosen weld size is sufficient."

Anchor Bolt Connection Design



WALL CAP TYPICAL DETAIL

Given:

Number of anchor bolts resisting loads: $n_{ab} := 2$ *Only one side's bolts resist tension or shear.*

Bolts are specified as ASTM F1554 and Grade A36

Bolt diameter: $d_{ab} := \frac{5}{8}\ in$ *Plans*

Bolt area: $A_b := 0.307\ in^2$ *AISC Tbl. 7-18*

Bolt nominal yield stress strength: $F_{y\_bolt} := 36\ ksi$ *AISC Tbl. 2-3*

Bolt nominal ultimate tensile stress strength: $F_{u\_bolt} := 58\ ksi$

Bolt embedment: $h_{ef} := 5\ in$ *Plans*

Output:

Tension anchor bolt spacing: $s_1 := \frac{B_{plate} - 2 \cdot x_{bolt}}{n_{ab} - 1}$ $s_1 = 7\ in$

Bolt nominal tensile stress strength: $F_{nt} := 0.75 \cdot F_{u\_bolt}$ $F_{nt} = 43.5\ ksi$ *AISC Tbl. J3.2*

Bolt nominal shear stress strength: $F_{nv} := 0.40 \cdot F_{u\_bolt}$ $F_{nv} = 23.2\ ksi$ *AISC Tbl. J3.2, assuming threads within shear plane*

Ultimate tension load on one anchor bolt: $T_{u\_ab} := \frac{T_u}{n_{ab}}$ $T_{u\_ab} = 1.121\ kip$

Required shear stress on one bolt: $f_v := \frac{V_{post\_u}}{n_{ab} \cdot A_b}$ $f_v = 0.57\ ksi$

Bolt modified nominal tensile stress strength, modified for effects of shear stress: $F_{nt}' := \min\left(F_{nt}, 1.3 \cdot F_{nt} - \frac{F_{nt}}{\phi_{ab} \cdot F_{nv}} \cdot f_v\right)$ $F_{nt}' = 43.5\ ksi$ *AISC Eqn. J3-3a*

Bolt factored tensile resistance: $\phi R_{n\_bolt} := \phi_{ab} \cdot F_{nt}' \cdot A_b$ $\phi R_{n\_bolt} = 10.016\ kip$ *AISC Eqn. J3-2*

Check of bolt tensile stress:

$Bolt\_Tensile\_Check :=$ if $\phi R_{n\_bolt} \geq T_{u\_ab}$
  "Bolt is satisfactory."
else
  "Bolt is no good."

$Bolt\_Tensile\_Check =$ "Bolt is satisfactory."

Continuing Anchor Bolt Connection Design per ACI 318

Outside diameter of anchor: $d_a := d_{ab}$ $d_a = 0.625\ in$

Critical edge distance for adhesive anchors: $c_{ac} := 2\ h_{ef}$ $c_{ac} = 10\ in$ *ACI 17.7.6*

Steel strength of anchor in tension (ACI 17.4.1)

Steel tension strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in tension (ACI 17.4.2)

Check bolt group action for tension concrete breakout: *ACI 17.2.1.1*

$Group\_Tension\_Breakout\_Check :=$ if $s_1 \leq 3 \cdot h_{ef}$
  "Bolts act in group."
else
  "Bolts act singly."

$Group\_Tension\_Breakout\_Check =$ "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Nco} := 9 \cdot h_{ef}^2$ $A_{Nco} = 225\ in^2$ *ACI Eqn. 17.4.2.1c*

Actual projected influence area for bolt(s): $A_{Nc} := \min\left(\left(c_{a1} + 1.5 \cdot h_{ef}\right) \cdot \left(1.5 \cdot h_{ef} + \min\left(s_1, 3 \cdot h_{ef}\right) + \min\left(1.5 \cdot h_{ef}, c_{a2}\right)\right), n_{ab} \cdot A_{Nco}\right)$ $A_{Nc} = 281.93\ in^2$ *ACI Fig. R17.4.2.1*

Concrete tension breakout strength coefficient: $k_c := 17$ *Value of 17 for post-installed anchors, per ACI 17.4.2.2*

Basic concrete tension breakout strength for single anchor: $N_b := k_c \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lbf$ $N_b = 12.021\ kip$ *ACI Eqn. 17.4.2.2a*

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_N} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.4.2.4*

Factor for anchor bolts near an edge: $\Psi_{ed\_N} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a1}}{1.5 \cdot h_{ef}}\right)$ $\Psi_{ed\_N} = 0.913$ *ACI Eqn. 17.4.2.5b*

Factor for anchor bolts in un-cracked concrete: $\Psi_{c\_N} := 1.4$ *Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.* *ACI 17.4.2.6*

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement: $\Psi_{cp\_N} := min\left(1.0, \max\left(\frac{c_{a1}}{c_{ac}}, \frac{1.5 \cdot h_{ef}}{c_{ac}}\right)\right)$ $\Psi_{cp\_N} = 0.75$ *ACI Eqn. 17.4.2.7b*

Nominal concrete tension breakout strength: $\phi N_{cbg} := \phi_{adh} \cdot \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ec\_N} \cdot \Psi_{ed\_N} \cdot \Psi_{c\_N} \cdot \Psi_{cp\_N} \cdot N_b$ $\phi N_{cbg} = 9.382\ kip$ *ACI Eqn. 17.4.2.1b*

Check of concrete tension breakout failure:

$$Concrete\_Tension\_Breakout\_Check := \text{if } \phi N_{cbg} \ge n_{ab} \cdot T_{u\_ab} \;\; \text{"Bolt is satisfactory." else "Bolt is no good."}$$

$Concrete\_Tension\_Breakout\_Check =$ "Bolt is satisfactory."

Pullout strength cast-in, post-installed expansion, or undercut anchor in tension (ACI 17.4.3)

Proposed anchors are post-installed adhesive, not headed studs or bolts, expansion anchors, or undercut anchors; so, no check is required.

Concrete side-face blowout strength of headed anchor in tension (ACI 17.4.4)

Proposed anchors are post-installed adhesive, not headed studs or bolts; so, no check is required.

Bond strength of adhesive anchor in tension (ACI 17.4.5)

Minimum bond stress for HY 200 Epoxy per HILTI ESR-3187:

$$\tau_{uncr\_HY\_200} := 0.65 \cdot \left(\frac{f'_c}{2500\ psi}\right)^{0.1} \cdot 2220\ psi = 1512.441\ psi$$

Per HILTI ESR-3187 Table 14, basic un-cracked bond strength is 2,220 psi; this value is factored by a straight 0.65 for either wet or dry installation conditions and by a small boost from concrete strength higher than 2,500 psi

Minimum bond stress for HIT-RE 500 Epoxy per HILTI ESR-3814:

$$\tau_{uncr\_HIT\_RE\_500} := 0.65 \cdot \left(\frac{f'_c}{2500\ psi}\right)^{0.15} \cdot 2210\ psi = 1541.429\ psi$$

Per HILTI ESR-3814 Table 12, basic un-cracked bond strength is 2,210 psi. This value is based on diamond coring and roughening afterwards; it is lower than being hammer-drilled with carbide bit. The socket must be roughened if coring with a diamond bit; this should be written on the plans. Factors are a straight 0.65 reduction factor independent of wet or dry concrete conditions during installation and a small boost for using concrete higher than 2,500 psi. The smaller factor for cracked concrete is used since no supplementary rebar is being provided; this also matches with reduction factor below.

Minimum bond stress strength: $\tau_{uncr} := min\left(\tau_{uncr\_HY\_200}, \tau_{uncr\_HIT\_RE\_500}\right)$ $\tau_{uncr} = 1512.441\ psi$

Distance to edge of project influence area: $c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100\ psi}}$ $c_{Na} = 7.329\ in$ *ACI Eqn. 17.4.5.1d*

Check if anchor bolts act in group for bond failure: *ACI 17.2.1.1*

$$Group\_Bond\_Failure\_Check := \text{if } s_1 \le 2\ c_{Na} \;\; \text{"Bolts act in group." else "Bolts act singly."}$$

$Group\_Bond\_Failure\_Check =$ "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Nao} := (2\ c_{Na})^2$ $A_{Nao} = 214.835\ in^2$ *ACI Eqn. 17.4.5.1c*

Actual projected influence area for bolt(s): $A_{Na} := min\left(\left(c_{Na} + min\left(s_1, 2 \cdot c_{Na}\right) + min\left(c_{Na}, c_{a2}\right)\right) \cdot \left(c_{a1} + c_{Na}\right), n_{ab} \cdot A_{Nao}\right)$ $A_{Na} = 273.826\ in^2$ *ACI Fig. R17.4.5.1*

Basic bond strength of adhesive anchor: $N_{ba} := \tau_{uncr} \cdot \pi \cdot d_a \cdot h_{ef}$ $N_{ba} = 14.848\ kip$ *ACI Eqn. 17.4.5.2*

Concrete is not light weight; so, lambda-a is set to 1.0; per ACI 17.4.5.2, un-cracked bond stress may be used.

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_Na} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.4.5.3*

Factor for anchor bolts near an edge: $\Psi_{ed\_Na} := min\left(1.0, 0.7 + 0.3 \cdot \dfrac{c_{a1}}{c_{Na}}\right)$ $\Psi_{ed\_Na} = 0.918$ *ACI Eqn. 17.4.5.4b*

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement: $\Psi_{cp\_Na} := min\left(1.0, \max\left(\dfrac{c_{a1}}{c_{ac}}, \dfrac{c_{Na}}{c_{ac}}\right)\right)$ $\Psi_{cp\_Na} = 0.733$ *ACI Eqn. 17.4.5.5b*

Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

Nominal bond strength of the adhesive anchor(s): $\phi N_{ag} := \phi_{adh} \cdot \dfrac{A_{Na}}{A_{Nao}} \cdot \Psi_{ec\_Na} \cdot \Psi_{ed\_Na} \cdot \Psi_{cp\_Na} \cdot N_{ba}$ $\phi N_{ag} = 8.272\ kip$ *ACI Eqn. 17.4.5.1b*

Check of bolt bond stress failure:

$$Bond\_Stress\_Check := \begin{cases} \text{if } \phi N_{ag} \ge n_{ab} \cdot T_{u\_ab} \\ \quad \text{“Bolt is satisfactory.”} \\ \text{else} \\ \quad \text{“Bolt is no good.”} \end{cases}$$

$Bond\_Stress\_Check = \text{“Bolt is satisfactory.”}$

Steel strength of anchor in shear (17.5.1)

Steel shear strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in shear (17.5.2)

Check bolt group action for shear concrete breakout: *ACI 17.2.1.1*

$$Group\_Shear\_Breakout\_Check := \begin{cases} \text{if } s_1 \le 3 \cdot c_{a1} \\ \quad \text{“Bolts act in group.”} \\ \text{else} \\ \quad \text{“Bolts act singly.”} \end{cases}$$

$Group\_Shear\_Breakout\_Check = \text{“Bolts act in group.”}$

Theoretical projected influence area of a single bolt far from an edge: $A_{Vco} := 4.5 \cdot {c_{a1}}^2$ $A_{Vco} = 127.122\ in^2$ *ACI Eqn. 17.5.2.1c*

Actual projected influence area for bolt(s): $A_{Vc} := min\left(1.5 \cdot c_{a1} \cdot \left(1.5 \cdot c_{a1} + min\left(s_1, 3 \cdot c_{a1}\right) + min\left(1.5 \cdot c_{a1}, c_{a2}\right)\right), n_{ab} \cdot A_{Vco}\right)$ $A_{Vc} = 182.929\ in^2$ *ACI Fig. R17.5.2.1b*

Load bearing length: $l_e := h_{ef}$ $l_e = 5\ in$ *ACI 17.5.2.2*

Basic concrete breakout strength in shear for single anchor:

$$V_b := min\left(\left(7\left(\frac{l_e}{d_a}\right)^{0.2} \cdot \sqrt{\frac{d_a}{in}}\right) \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5}, 9 \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5}\right) \cdot lbf$$

$V_b = 6.5\ kip$ *Concrete is not light weight; so, lambda-a is set to 1.0.* *ACI Eqns. 17.5.2.2a & 17.5.2.2b*

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_V} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.5.2.5*

Factor for anchor bolts near an edge: $\Psi_{ed\_V} := min\left(1.0, 0.7 + 0.3 \cdot \dfrac{c_{a2}}{1.5 \cdot c_{a1}}\right)$ $\Psi_{ed\_V} = 1$ *ACI Eqns. 17.5.2.6a & 17.5.2.6b*

Factor for anchor bolts in un-cracked concrete: $\Psi_{c\_V} := 1.4$ *Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.* *ACI 17.5.2.7*

Factor for small embedment $\Psi_{h\_V} := min\left(1.0, \sqrt{\frac{1.5 \cdot c_{a1}}{h_{ef}}}\right)$ $\Psi_{h\_V} = 1$ *ACI Eqn. 17.5.2.8*

Nominal concrete shear breakout strength: $\phi V_{cbg} := \phi_{adh} \cdot \frac{A_{Vc}}{A_{Vco}} \cdot \Psi_{ec\_V} \cdot \Psi_{ed\_V} \cdot \Psi_{c\_V} \cdot \Psi_{h\_V} \cdot V_b$ $\phi V_{cbg} = 8.512$ ***kip*** *ACI Eqn. 17.5.2.1b*

Check of concrete shear breakout failure: $Concrete\_Shear\_Breakout\_Check :=$ if $\phi V_{cbg} \geq V_{post\_u}$
"Bolt is satisfactory."
else
"Bolt is no good."

$Concrete\_Shear\_Breakout\_Check =$ "Bolt is satisfactory."

Concrete pryout strength of anchor in shear (17.5.3)

Basic concrete pryout strength of a single anchor in shear: $\phi N_{cpg} := min\left(\phi N_{ag}, \phi N_{cbg}\right)$ $\phi N_{cpg} = 8.272$ ***kip*** *ACI 17.5.3.1*

Concrete pryout strength in shear coefficient: $k_{cp} :=$ if $h_{ef} < 2.5$ ***in***
1.0
else
2.0

$k_{cp} = 2$ *ACI 17.5.3.1*

Nominal concrete pryout strength of anchor(s) in shear: $\phi V_{cpg} := k_{cp} \cdot \phi N_{cpg}$ $\phi V_{cpg} = 16.545$ ***kip*** *ACI Eqn. 17.5.3.1b*

Check of concrete pryout strength in shear: $Concrete\_Shear\_Pryout\_Check :=$ if $\phi V_{cpg} \geq V_{post\_u}$
"Bolt is satisfactory."
else
"Bolt is no good."

$Concrete\_Shear\_Pryout\_Check =$ "Bolt is satisfactory."

Vertical Interior Post and Horizontal Rail Design

Given:

Post Height: $H_{post} := 66\ in$ Plans

Step Height: $H_{step} := 24\ in$

Post and Rail Yield Strength: $F_y := 50\ ksi$ ASTM F1043

Post and Rail Modulus of Elasticity: $E_s := 29000\ ksi$

Post and Rail Ultimate Strength: $F_u := 58\ ksi$

Properties for ASTM F1043 IC 2-3/8" Pipe for Interior Posts

Post OD: $OD_{post} := 2.375\ in$

Post Thickness: $t_{post} := 0.13\ in$

Properties for ASTM F1043 IC 1-5/8" Pipe for Rails

Rail OD: $OD_{rail} := 1.66\ in$

Rail Thickness: $t_{rail} := 0.111\ in$

Design Point Live Load $P_{LL} := 200\ lbf$ AASHTO 13.8.2

Design Uniform Live Load $w_{LL} := 0\ plf$ AASHTO 13.8.2

Post spacing: $L_{spc} := 8\ ft$ Plans

Weight of chain link fence: $f_{clf} := 0.48\ psf$

Design wind load from chain link fence: $f_{wind} := 15\ psf$ AASHTO 13.8.2

Load Factors (AASHTO Tbl. 3.4.1-1):

PL Load Factor: $\gamma_{PL} := 1.75$

DC Load Factor: $\gamma_{DL} := 1.25$

WS Load Factor: $\gamma_{WS} := 1.00$

Resistance Factors:

Steel Flexure (AASHTO 6.5.4.2): $\phi_f := 1.00$

Steel Shear (AASHTO 6.5.4.2): $\phi_v := 1.00$

Tension, Yielding in Gross Section: $\phi_y := 0.95$

Bending (AISC F1): $\phi_b := 0.90$

Shear (AISC G1): $\phi_{v\_AISC} := 0.90$

Bearing (AISC DG#1): $\phi_{brg} := 0.60$

Fillet Weld (AISC Tbl. J2.5): $\phi_{fw} := 0.75$

Bolts (AISC J3.6 & J3.7): $\phi_{ab} := 0.75$

Adhesive Anchor Bolts (ACI 17.3.3, Condition B, Category 1): $\phi_{adh} := 0.65$

Steel weight density: $\gamma_{steel} := 490\ pcf$

ASTM F1043 Group IC Electric Resistant Welded 50,000 psi yield steel pipe																	
Trade Reference	Decimal O.D. Equivalent		Pipe wall Thickness		Weight		Section Modulus		x	Min. Yield Strength		=	Max Bending Moment	Calculated Load (lbs)		
														10' Free	Cantilever	
O.D.	inches	(mm)	inches	(mm)	lb./ft.	(kg/m)	inches³	(mm³)	x	psi	(Mpa)	=	Lb.In.	Supported	4'	6'
1 5/8"	1.660	42.16	0.111	2.82	1.84	2.74	0.1962	4.98	x	50000	345	=	9810	327	204	136
1 7/8"	1.900	48.26	0.120	3.05	2.28	3.39	0.2810	7.14	x	50000	345	=	14050	468	293	195
2 3/8"	2.375	60.33	0.130	3.30	3.12	4.64	0.4881	12.40	x	50000	345	=	24405	814	508	339
2 7/8"	2.875	73.03	0.160	4.06	4.64	6.91	0.8778	22.30	x	50000	345	=	43890	1463	914	610
3 1/2"	3.500	88.90	0.160	4.06	5.71	8.50	1.3408	34.06	x	50000	345	=	67042	2235	1397	931
4"	4.000	101.60	0.160	4.06	6.57	9.78	1.7820	45.26	x	50000	345	=	89098	2970	1856	1237
4 1/2"	4.500	114.30	0.160	4.14	7.42	11.04	2.2859	57.99	X	50000	345	=	114295	3810	5486	1587

Output:

Post Section Properties:

Post inside diameter: $ID_{post} := OD_{post} - 2 \cdot t_{post}$ $ID_{post} = 2.115\ in$

Post Area: $A_{post} := 0.785398 \cdot \left(OD_{post}^2 - ID_{post}^2\right)$ $A_{post} = 0.917\ in^2$

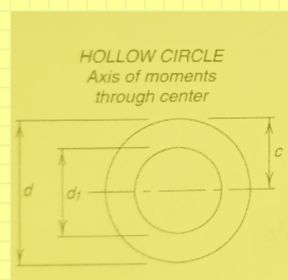
Post Unit Weight: $w_{post} := \gamma_{steel} \cdot A_{post}$ $w_{post} = 3.12\ plf$

Post centroid: $c_{post} := 0.5 \cdot OD_{post}$ $c_{post} = 1.188\ in$

Post Moment of Inertial: $I_{post} := 0.049087 \cdot \left(OD_{post}^4 - ID_{post}^4\right)$ $I_{post} = 0.58\ in^4$

Post Section Modulus: $S_{post} := \dfrac{I_{post}}{c_{post}}$ $S_{post} = 0.488\ in^3$

Post Plastic Section Modulus: $Z_{post} := \dfrac{OD_{post}^3 - ID_{post}^3}{6}$ $Z_{post} = 0.656\ in^3$



$$A = \frac{\pi(d^2 - d_1^2)}{4} = .785398\,(d^2 - d_1^2)$$

$$c = \frac{d}{2}$$

$$I = \frac{\pi(d^4 - d_1^4)}{64} = .049087\,(d^4 - d_1^4)$$

$$S = \frac{\pi(d^4 - d_1^4)}{32d} = .098175\ \frac{d^4 - d_1^4}{d}$$

$$r = \frac{\sqrt{d^2 + d_1^2}}{4}$$

$$Z = \frac{d^3}{6} - \frac{d_1^3}{6}$$

Rail Section Properties:

Rail inside diameter: $ID_{rail} := OD_{rail} - 2 \cdot t_{rail}$ $ID_{rail} = 1.438\ in$

Rail Area: $A_{rail} := 0.785398 \cdot \left(OD_{rail}^2 - ID_{rail}^2\right)$ $A_{rail} = 0.54\ in^2$

Rail Unit Weight: $w_{rail} := \gamma_{steel} \cdot A_{rail}$ $w_{rail} = 1.838\ plf$

Rail centroid: $c_{rail} := 0.5 \cdot OD_{rail}$ $c_{rail} = 0.83\ in$

Rail Moment of Inertial: $I_{rail} := 0.049087 \cdot \left(OD_{rail}^4 - ID_{rail}^4\right)$ $I_{rail} = 0.163\ in^4$

Rail Section Modulus: $S_{rail} := \dfrac{I_{rail}}{c_{rail}}$ $S_{rail} = 0.196\ in^3$

Rail Plastic Section Modulus: $Z_{rail} := \dfrac{OD_{rail}^3 - ID_{rail}^3}{6}$ $Z_{rail} = 0.267\ in^3$

Post concentrated live load applied at high top rail: $P_{post\_LL\_H} := P_{LL} + w_{LL} \cdot \frac{L_{spc}}{2} = 0.2\ kip$ $P_{post\_LL\_H} = 0.2\ kip$ *AASHTO Eqn. 13.8.2-1, modified for split top rails*

Post concentrated live load applied at low top rail: $P_{post\_LL\_L} := w_{LL} \cdot \frac{L_{spc}}{2} = 0\ kip$ $P_{post\_LL\_L} = 0\ kip$

Post moment loading from live load: $M_{post\_LL} := P_{post\_LL\_H} \cdot H_{post} + P_{post\_LL\_L} \cdot \left(H_{post} - H_{step}\right)$ $M_{post\_LL} = 13200\ lbf \cdot in$ *Post treated as cantilevered beam*

Post shear from live load: $V_{post\_LL} := P_{post\_LL\_H} + P_{post\_LL\_L}$ $V_{post\_LL} = 0.2\ kip$

Rail moment from live load applied: $M_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}^{\ 2}}{8} + \frac{P_{LL} \cdot L_{spc}}{4}$ $M_{rail\_LL} = 4800\ lbf \cdot in$ *Rail treated as simply supported beam with vertical and horizontal live loads combined into resultant direction.*

Rail moment from dead load: $M_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}^{\ 2}}{8} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}^{\ 2}}{8}$ $M_{rail\_DL} = 303.173\ lbf \cdot in$

Rail shear from live load: $V_{rail\_LL} := \sqrt{2} \cdot \frac{w_{LL} \cdot L_{spc}}{2} + \frac{P_{LL}}{2}$ $V_{rail\_LL} = 0.1\ kip$

Rail shear from dead load: $V_{rail\_DL} := \frac{w_{rail} \cdot L_{spc}}{2} + \frac{f_{clf} \cdot \frac{H_{post}}{2} \cdot L_{spc}}{2}$ $V_{rail\_DL} = 0.013\ kip$

Factored Shear Load on Post: $V_{post\_u} := \gamma_{PL} \cdot V_{post\_LL}$ $V_{post\_u} = 0.35\ kip$

Factored Moment Load on Post: $M_{post\_u} := \gamma_{PL} \cdot M_{post\_LL}$ $M_{post\_u} = 23100\ lbf \cdot in$

AASHTO load factors used instead of ASCE load factors found in AISC and ACI. This is acceptable as it is more conservative.

Factored Shear Load on Rail: $V_{rail\_u} := \gamma_{PL} \cdot V_{rail\_LL} + \gamma_{DL} \cdot V_{rail\_DL}$ $V_{rail\_u} = 0.191\ kip$

Factored Moment Load on Rail: $M_{rail\_u} := \gamma_{PL} \cdot M_{rail\_LL} + \gamma_{DL} \cdot M_{rail\_DL}$ $M_{rail\_u} = 8778.966\ lbf \cdot in$

Vertical dead load was combined directly with live load resultant since it was so small compared to the live load.

Post Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area: $A_g := A_{post}$ $A_g = 0.917\ in^2$

Distance from Max to 0 Shear: $L_v := H_{post}$ $L_v = 66\ in$

Critical Strength for Shear:

$$F_{cr} := min\left(0.58 \cdot F_y, \max\left(\frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{post}}} \left(\frac{OD_{post}}{t_{post}}\right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left(\frac{OD_{post}}{t_{post}}\right)^{\frac{3}{2}}}\right)\right)$$

$F_{cr} = 29\ ksi$ *AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3*

Factored nominal shear resistance: $\phi V_n := \phi_v \cdot 0.5\ F_{cr} \cdot A_g$ $\phi V_n = 13.295\ kip$ *AASHTO Eqn. 6.12.1.2.3c-1*

Post Shear Check: $\frac{\phi V_n}{V_{post\_u}} = 37.985$

$Post\_Shear\_Check :=$ if $\frac{\phi V_n}{V_{post\_u}} \geq 1.0$
“Post shear strength is satisfactory.”
else
“Post is not satisfactory.”

$Post\_Shear\_Check =$ “Post shear strength is satisfactory.”

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

$Check\_Compact :=$ if $\frac{OD_{post}}{t_{post}} \leq \frac{0.07 \cdot E_s}{F_y}$
“Section is compact. Local buckling does not apply.”
else
“Section is not compact. Check wall slenderness.”

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

$Check\_Compact =$ “Section is compact. Local buckling does not apply.”

Factored Nominal Moment Resistance: $\phi M_n := \phi_f \cdot F_y \cdot Z_{post}$ $\phi M_n = 32.797\ kip \cdot in$ *AASHTO Eqn. 6.12.2.2.3-1*

Post Flexural Check: $\dfrac{\phi M_n}{M_{post\_u}} = 1.42$

$Post\_Flex\_Check :=$ if $\dfrac{\phi M_n}{M_{post\_u}} \ge 1.0$
  "Post flexural strength is satisfactory."
else
  "Post is not satisfactory."

$Post\_Flex\_Check =$ "Post flexural strength is satisfactory."

Rail Analysis:

Following AASHTO 6.12.1.2.3c for Shear Design:

Gross Area: $A_g := A_{rail}$ $A_g = 0.54\ in^2$

Distance from Max to 0 Shear: $L_v := \dfrac{L_{spc}}{2}$ $L_v = 48\ in$

Critical Strength for Shear:

$$F_{cr} := min\left(0.58 \cdot F_y, \max\left(\frac{1.6 \cdot E_s}{\sqrt{\frac{L_v}{OD_{rail}}}\left(\frac{OD_{rail}}{t_{rail}}\right)^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{\left(\frac{OD_{rail}}{t_{rail}}\right)^{\frac{3}{2}}}\right)\right)$$

$F_{cr} = 29\ ksi$ *AASHTO Eqns. 6.12.1.2.3c-2 & 6.12.1.2.3c-3*

Factored Nominal Shear Resistance: $\phi V_n := \phi_v \cdot 0.5\ F_{cr} \cdot A_g$ $\phi V_n = 7.832\ kip$ *AASHTO Eqn. 6.12.1.2.3c-1*

Rail Shear Check: $\dfrac{\phi V_n}{V_{rail\_u}} = 41.052$

$Rail\_Shear\_Check :=$ if $\dfrac{\phi V_n}{V_{rail\_u}} \ge 1.0$
  "Rail shear strength is satisfactory."
else
  "Rail is not satisfactory."

$Rail\_Shear\_Check =$ "Rail shear strength is satisfactory."

Following AASHTO 6.12.2.2.3 for Flexure Design:

Check of Noncompact Section:

$Check\_Compact :=$ if $\dfrac{OD_{rail}}{t_{rail}} \le \dfrac{0.07 \cdot E_s}{F_y}$
  "Section is compact. Local buckling does not apply."
else
  "Section is not compact. Check wall slenderness."

Per AASHTO 6.12.2.2.3, as long D/t does not exceed 0.07E/Fy, plastic modulus and equation 6.12.2.2.3-1 may be used.

$Check\_Compact =$ "Section is compact. Local buckling does not apply."

Factored Nominal Moment Resistance: $\phi M_n := \phi_f \cdot F_y \cdot Z_{rail}$ $\phi M_n = 13.339\ kip \cdot in$ *AASHTO Eqn. 6.12.2.2.3-1*

Post Flexural Check: $\dfrac{\phi M_n}{M_{rail\_u}} = 1.519$

$Rail\_Flex\_Check :=$ if $\dfrac{\phi M_n}{M_{rail\_u}} \ge 1.0$
  "Rail flexural strength is satisfactory."
else
  "Rail is not satisfactory."

$Rail\_Flex\_Check =$ "Rail flexural strength is satisfactory."

Confirming that Wind Loading Doesn't Control:

Per last paragraph of AASHTO 13.8.2, the wind load on the chain link fence is not applied simultaneously with the live load.

Uniform wind load on post: $w_{post\_wind} := f_{wind} \cdot L_{spc}$ $w_{post\_wind} = 120\ plf$

Design moment from wind on post: $M_{post\_wind\_u} := \gamma_{WS} \cdot \frac{w_{post\_wind} \cdot H_{post}^{2}}{2}$ $M_{post\_wind\_u} = 21780\ lbf \cdot in$

$M_{post\_u} = 23100\ lbf \cdot in$ <- LL controls

Design shear from wind on post: $V_{post\_wind\_u} := \gamma_{WS} \cdot w_{post\_wind} \cdot H_{post}$ $V_{post\_wind\_u} = 0.66\ kip$

$V_{post\_u} = 0.35\ kip$ <- LL controls

Uniform wind on rail: $w_{rail\_wind} := f_{wind} \cdot \frac{H_{post}}{2}$ $w_{rail\_wind} = 41.25\ plf$

Design moment from wind on rail: $M_{rail\_wind\_u} := \gamma_{WS} \cdot \frac{w_{rail\_wind} \cdot L_{spc}^{2}}{8}$ $M_{rail\_wind\_u} = 3960\ lbf \cdot in$

$M_{rail\_u} = 8778.966\ lbf \cdot in$ <- LL controls

Design shear from wind on rail: $V_{rail\_wind\_u} := \gamma_{WS} \cdot w_{rail\_wind} \cdot \frac{L_{spc}}{2}$ $V_{rail\_wind\_u} = 0.165\ kip$

$V_{rail\_u} = 0.191\ kip$ <- LL controls

Base Plate Design - Line Post w/ Axial Compression

Given: *Plans*

Cap width: $W_{cap} := 15.63\ in$

Distance from post to end of cap: $L_{end} := 8\ in$

Plate thickness: $t_p := .5\ in$

Plate length (perpendicular to fence): $N_{plate} := 8\ in$

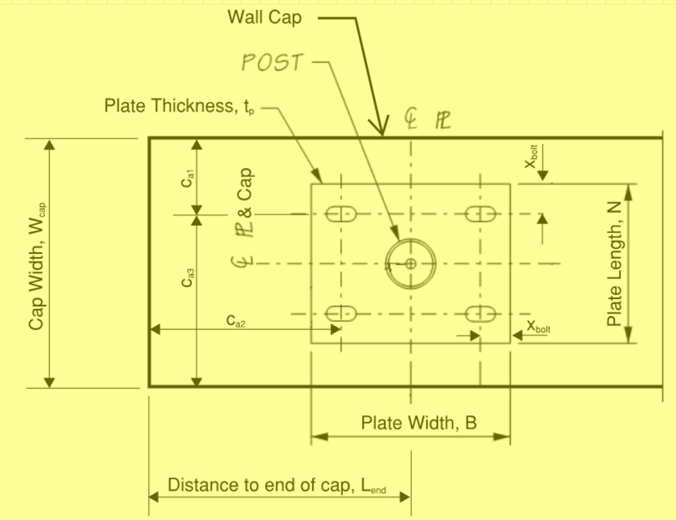
Plate width (parallel to fence): $B_{plate} := 10\ in$

Compressive Strength of Concrete: $f'_c := 4\ ksi$

Side clearance to anchor bolts: $x_{bolt} := 1.5\ in$

Base plate steel yield strength: $F_{y\_plate} := 36\ ksi$

Number of rails: $n_{rail} := 4$



Output:

Plate Area: $A_{plate} := N_{plate} \cdot B_{plate}$ $\quad A_{plate} = 80\ in^2$

Distance from bolt to near face of cap: $c_{a1} := \frac{1}{2}\left(W_{cap} - N_{plate}\right) + x_{bolt}$ $\quad c_{a1} = 5.315\ in$

Distance from outside bolt to end of cap: $c_{a2} := L_{end} - \frac{B_{plate}}{2} + x_{bolt}$ $\quad c_{a2} = 4.5\ in$

Distance from bolt to far face of cap: $c_{a3} := W_{cap} - c_{a1}$ $\quad c_{a3} = 10.315\ in$

Bearing Area taken to Be Same as Plate Area: $A_{bearing} := A_{plate}$ $\quad A_{bearing} = 80\ in^2$ *Conservatively setting bearing area to the same as the plate.*

Max allowed bearing pressure: $f_{pu\_max} := \phi_{brg} \cdot min\left(0.85 \cdot f'_c \cdot \sqrt{\frac{A_{bearing}}{A_{plate}}}, 1.7 \cdot f'_c\right)$ $\quad f_{pu\_max} = 2.04\ ksi$ *ACI Tbl. 14.5.6.1*

Max allowed bearing pressure line: $q_{max} := f_{pu\_max} \cdot B_{plate}$ $\quad q_{max} = \left(2.448 \cdot 10^5\right) \frac{lbf}{ft}$

Post dead load on plate $P_{post\_DL} := w_{post} \cdot H_{post}$ $\quad P_{post\_DL} = 0.017\ kip$

Rail dead load on plat: $P_{rail\_DL} := n_{rail} \cdot 2 \cdot V_{rail\_DL}$ $\quad P_{rail\_DL} = 0.101\ kip$

Factored vertical load on plate: $P_u := \gamma_{DL} \cdot \left(P_{post\_DL} + P_{rail\_DL}\right)$ $\quad P_u = 0.148\ kip$

Minimum length of area of bearing: $Y_{min} := \frac{P_u}{q_{max}}$ $\quad Y_{min} = 0.007\ in$ *AISC DG#1 Eqn. 3.3.3*

Critical eccentricity distance: $e_{crit} := \frac{N_{plate}}{2} - \frac{Y_{min}}{2}$ $\quad e_{crit} = 3.996\ in$ *AISC DG#1 Eqn. 3.3.7*

Eccentricity of loading: $e_{loading} := \frac{M_{post\_u}}{P_u}$ $\quad e_{loading} = 156.322\ in$ *AISC DG#1 Eqn. 3.3.6*

Small moment check: $Small\_Moment\_Check :=$ if $e_{loading} \le e_{crit}$
‖ "Moment is small, no need for anchor bolts."
else
‖ "Moment is large, need anchor bolts."

$Small\_Moment\_Check =$ "Moment is large, need anchor bolts."

Distance from bolt to center of post: $f_{dim} := \frac{N_{plate}}{2} - x_{bolt}$ $\quad f_{dim} = 2.5\ in$

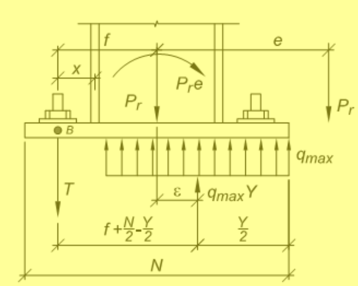


Fig. 3.4.1. Base plate with large moment.

Plate dimension check

$$Plate\_Dim\_Check := \text{if } \left(f_{dim} + \frac{N_{plate}}{2}\right)^2 \ge \frac{2 \cdot P_u \cdot \left(e_{loading} + f_{dim}\right)}{q_{max}}$$
$$\quad \| \text{“Plate dimensions are OK.”}$$
$$\text{else}$$
$$\quad \| \text{“Plate needs to be longer and/or wider.”}$$

$Plate\_Dim\_Check =$ “Plate dimensions are OK.”

Length of bearing area centered at the eccentricity of this loading:

$$Y_{loading} := \left(f_{dim} + \frac{N_{plate}}{2}\right) - \sqrt{\left(f_{dim} + \frac{N_{plate}}{2}\right)^2 - \frac{2 \cdot P_u \cdot \left(e_{loading} + f_{dim}\right)}{q_{max}}}$$

$Y_{loading} = 0.179\ \boldsymbol{in}$

AISC DG#1 Eqn. 3.4.3

Required tensile resistance in anchor rods: $T_u := q_{max} \cdot Y_{loading} - P_u$ $T_u = 3.513\ \boldsymbol{kip}$ *AISC DG#1 Eqn. 3.4.2*

Find minimum required thickness for plate based on bending at bearing interface:

Find bearing bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$m_{plate} := \frac{N_{plate} - 0.8\ OD_{post}}{2}$$

$m_{plate} = 3.05\ \boldsymbol{in}$

Calculating minimum thickness based on bearing:

$$t_{p\_brng\_req} := \text{if } Y_{loading} \ge m_{plate}$$
$$\quad \| 1.5 \cdot m_{plate} \cdot \sqrt{\frac{f_{pu\_max}}{F_{y\_plate}}}$$
$$\text{else}$$
$$\quad \| 2.11 \cdot \sqrt{\frac{f_{pu\_max} \cdot Y_{loading} \cdot \left(m_{plate} - \frac{Y_{loading}}{2}\right)}{F_{y\_plate}}}$$

$t_{p\_brng\_req} = 0.366\ \boldsymbol{in}$

AISC DG#1 Eqns. 3.3.14a-2 & 3.3.15a-2

Find minimum required thickness for plate based on bending at tension interface:

Find tension bending line distance from edge of plate (AISC DG#1, 3.1.3):

$$x_{ten} := f_{dim} - \frac{0.8 \cdot OD_{post}}{2}$$

$x_{ten} = 1.55\ \boldsymbol{in}$

Calculating minimum thickness based on tension:

$$t_{p\_ten\_req} := 2.11 \cdot \sqrt{\frac{T_u \cdot x_{ten}}{B_{plate} \cdot F_{y\_plate}}}$$

$t_{p\_ten\_req} = 0.26\ \boldsymbol{in}$ *AISC DG#1 Eqn. 3.4.7a*

Controlling minimum required base plate thickness: $t_{p\_req} := \max\left(t_{p\_brng\_req}, t_{p\_ten\_req}\right)$ $t_{p\_req} = 0.366\ \boldsymbol{in}$

Check chosen plate thickness:

$$Plate\_Thick\_Check := \text{if } t_p \ge t_{p\_req}$$
$$\quad \| \text{“Chosen plate thickness is adequate.”}$$
$$\text{else}$$
$$\quad \| \text{“Need a thicker plate.”}$$

$Plate\_Thick\_Check =$ “Chosen plate thickness is adequate.”

Pipe to Plate Fillet Weld Connection Design

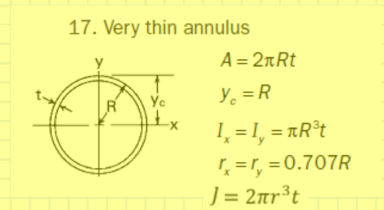
Given:

Minimum Fillet Weld Size:	$w_{min} := \frac{1}{8}\ in$	*Min fillet weld size based on AISC Table J2-4*
Chosen fillet weld size	$w := \frac{5}{16}\ in$	
Weld material:	$F_{EXX} := 70\ ksi$	

Output:

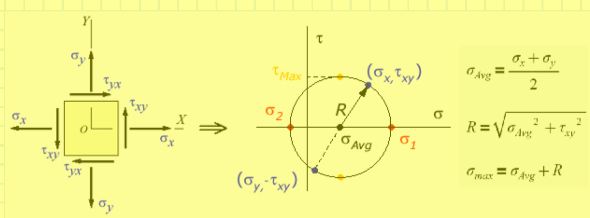
Welded Connection to Base Plate Design:

Gross Length of Weld is Post Perimeter:	$L_g := \pi \cdot OD_{post}$	$L_g = 7.461\ in$	
Effective Length of Weld:	$L_w := L_g - 2 \cdot w$	$L_w = 6.836\ in$	
Effective Throat Thickness:	$t_e := min\left(w \cdot \sin(45\ deg), \frac{L_w}{4}\right)$	$t_e = 0.221\ in$	*AISC, Sect. J2, Pts. 2a & 2b*
Area of Weld:	$A_w := L_w \cdot t_e$	$A_w = 1.511\ in^2$	*AISC, Sect. J2, Pts. 2a*
Moment of Inertia of circular fillet weld:	$I_w := \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e$	$I_w = 1.162\ in^4$	
Polar moment of Inertia of circular fillet weld:	$J_w := 2\ \pi \cdot \left(\frac{OD_{post}}{2}\right)^3 \cdot t_e$	$J_w = 2.325\ in^4$	



Determine design strength of weld:

Nominal strength of weld metal:	$F_w := \phi_{fw} \cdot 0.6 \cdot F_{EXX}$	$F_w = 31.5\ ksi$	*AISC, Tbl. J2.5*
Normal stress caused by bending moment:	$\sigma_b := \frac{M_{post\_u} \cdot \left(\frac{OD_{post}}{2}\right)}{I_w}$	$\sigma_b = 23.597\ ksi$	$\sigma = \frac{M}{S} = \frac{M \cdot c}{I}$
Stress caused by shearing force:	$\tau_v := \frac{V_{post\_u}}{A_w}$	$\tau_v = 0.232\ ksi$	
Resultant stress in weld from loading:	$\sigma_{max} := \frac{\sigma_b}{2} + \sqrt{\left(\frac{\sigma_b}{2}\right)^2 + \tau_v^{\ 2}}$	$\sigma_{max} = 23.599\ ksi$	

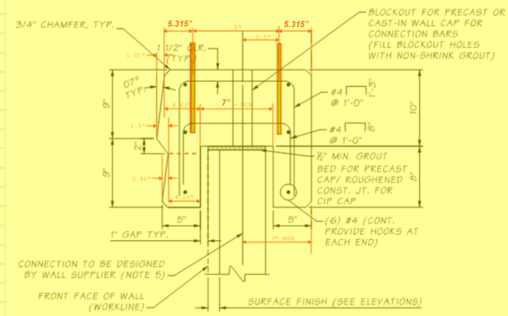


Check of weld thickness:

$$Weld\_Design\_Check := \begin{array}{l} \text{if } F_w \ge \sigma_{max} \\ \quad \|\ \text{“Chosen weld size is sufficient.”} \\ \text{else} \\ \quad \|\ \text{“Need bigger fillet weld.”} \end{array}$$

$$Weld\_Design\_Check = \text{“Chosen weld size is sufficient.”}$$

Anchor Bolt Connection Design



WALL CAP TYPICAL DETAIL

Given:

Number of anchor bolts resisting loads: $n_{ab} := 2$ *Only one side's bolts resist tension or shear.*

Bolts are specified as ASTM F1554 and Grade A36

Bolt diameter: $d_{ab} := \frac{5}{8}\ in$ *Plans*

Bolt area: $A_b := 0.307\ in^2$ *AISC Tbl. 7-18*

Bolt nominal yield stress strength: $F_{y\_bolt} := 36\ ksi$ *AISC Tbl. 2-3*

Bolt nominal ultimate tensile stress strength: $F_{u\_bolt} := 58\ ksi$

Bolt embedment: $h_{ef} := 5\ in$ *Plans*

Output:

Tension anchor bolt spacing: $s_1 := \frac{B_{plate} - 2 \cdot x_{bolt}}{n_{ab} - 1}$ $s_1 = 7\ in$

Bolt nominal tensile stress strength: $F_{nt} := 0.75 \cdot F_{u\_bolt}$ $F_{nt} = 43.5\ ksi$ *AISC Tbl. J3.2*

Bolt nominal shear stress strength: $F_{nv} := 0.40 \cdot F_{u\_bolt}$ $F_{nv} = 23.2\ ksi$ *AISC Tbl. J3.2, assuming threads within shear plane*

Ultimate tension load on one anchor bolt: $T_{u\_ab} := \frac{T_u}{n_{ab}}$ $T_{u\_ab} = 1.757\ kip$

Required shear stress on one bolt: $f_v := \frac{V_{post\_u}}{n_{ab} \cdot A_b}$ $f_v = 0.57\ ksi$

Bolt modified nominal tensile stress strength, modified for effects of shear stress: $F_{nt}' := min\left(F_{nt}, 1.3 \cdot F_{nt} - \frac{F_{nt}}{\phi_{ab} \cdot F_{nv}} \cdot f_v\right)$ $F_{nt}' = 43.5\ ksi$ *AISC Eqn. J3-3a*

Bolt factored tensile resistance: $\phi R_{n\_bolt} := \phi_{ab} \cdot F_{nt}' \cdot A_b$ $\phi R_{n\_bolt} = 10.016\ kip$ *AISC Eqn. J3-2*

Check of bolt tensile stress:

$Bolt\_Tensile\_Check :=$ if $\phi R_{n\_bolt} \geq T_{u\_ab}$
  "Bolt is satisfactory."
else
  "Bolt is no good."

$Bolt\_Tensile\_Check =$ "Bolt is satisfactory."

Continuing Anchor Bolt Connection Design per ACI 318

Outside diameter of anchor: $d_a := d_{ab}$ $d_a = 0.625\ in$

Critical edge distance for adhesive anchors: $c_{ac} := 2\ h_{ef}$ $c_{ac} = 10\ in$ *ACI 17.7.6*

Steel strength of anchor in tension (ACI 17.4.1)

Steel tension strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in tension (ACI 17.4.2)

Check bolt group action for tension concrete breakout: *ACI 17.2.1.1*

$Group\_Tension\_Breakout\_Check :=$ if $s_1 \leq 3 \cdot h_{ef}$
  "Bolts act in group."
else
  "Bolts act singly."

$Group\_Tension\_Breakout\_Check =$ "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Nco} := 9 \cdot h_{ef}^2$ $A_{Nco} = 225\ in^2$ *ACI Eqn. 17.4.2.1c*

Actual projected influence area for bolt(s): $A_{Nc} := min\left(\left(c_{a1} + 1.5 \cdot h_{ef}\right) \cdot \left(1.5 \cdot h_{ef} + min\left(s_1, 3 \cdot h_{ef}\right) + min\left(1.5 \cdot h_{ef}, c_{a2}\right)\right), n_{ab} \cdot A_{Nco}\right)$ $A_{Nc} = 243.485\ in^2$ *ACI Fig. R17.4.2.1*

Concrete tension breakout strength coefficient: $k_c := 17$ *Value of 17 for post-installed anchors, per ACI 17.4.2.2*

Basic concrete tension breakout strength for single anchor: $N_b := k_c \cdot 1.0 \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lbf$ $\quad N_b = 12.021\ kip$ *ACI Eqn. 17.4.2.2a*

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_N} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.4.2.4*

Factor for anchor bolts near an edge: $\Psi_{ed\_N} := min\left(1.0, 0.7 + 0.3 \cdot \frac{c_{a1}}{1.5 \cdot h_{ef}}\right)$ $\quad \Psi_{ed\_N} = 0.913$ *ACI Eqn. 17.4.2.5b*

Factor for anchor bolts in un-cracked concrete: $\Psi_{c\_N} := 1.4$ *Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.* *ACI 17.4.2.6*

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement: $\Psi_{cp\_N} := min\left(1.0, \max\left(\frac{c_{a1}}{c_{ac}}, \frac{1.5 \cdot h_{ef}}{c_{ac}}\right)\right)$ $\quad \Psi_{cp\_N} = 0.75$ *ACI Eqn. 17.4.2.7b*

Nominal concrete tension breakout strength: $\phi N_{cbg} := \phi_{adh} \cdot \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ec\_N} \cdot \Psi_{ed\_N} \cdot \Psi_{c\_N} \cdot \Psi_{cp\_N} \cdot N_b$ $\quad \phi N_{cbg} = 8.102\ kip$ *ACI Eqn. 17.4.2.1b*

Check of concrete tension breakout failure:

Concrete_Tension_Breakout_Check := if $\phi N_{cbg} \geq n_{ab} \cdot T_{u\_ab}$
  "Bolt is satisfactory."
else
  "Bolt is no good."

Concrete_Tension_Breakout_Check = "Bolt is satisfactory."

Pullout strength cast-in, post-installed expansion, or undercut anchor in tension (ACI 17.4.3)

Proposed anchors are post-installed adhesive, not headed studs or bolts, expansion anchors, or undercut anchors; so, no check is required.

Concrete side-face blowout strength of headed anchor in tension (ACI 17.4.4)

Proposed anchors are post-installed adhesive, not headed studs or bolts; so, no check is required.

Bond strength of adhesive anchor in tension (ACI 17.4.5)

Minimum bond stress for HY 200 Epoxy per HILTI ESR-3187:

$$\tau_{uncr\_HY\_200} := 0.65 \cdot \left(\frac{f'_c}{2500\ psi}\right)^{0.1} \cdot 2220\ psi = 1512.441\ psi$$

Per HILTI ESR-3187 Table 14, basic un-cracked bond strength is 2,220 psi; this value is factored by a straight 0.65 for either wet or dry installation conditions and by a small boost from concrete strength higher than 2,500 psi

Minimum bond stress for HIT-RE 500 Epoxy per HILTI ESR-3814:

$$\tau_{uncr\_HIT\_RE\_500} := 0.65 \cdot \left(\frac{f'_c}{2500\ psi}\right)^{0.15} \cdot 2210\ psi = 1541.429\ psi$$

Per HILTI ESR-3814 Table 12, basic un-cracked bond strength is 2,210 psi. This value is based on diamond coring and roughening afterwards; it is lower than being hammer-drilled with carbide bit. The socket must be roughened if coring with a diamond bit; this should be written on the plans. Factors are a straight 0.65 reduction factor independent of wet or dry concrete conditions during installation and a small boost for using concrete higher than 2,500 psi. The smaller factor for cracked concrete is used since no supplementary rebar is being provided; this also matches with reduction factor below.

Minimum bond stress strength: $\tau_{uncr} := min(\tau_{uncr\_HY\_200}, \tau_{uncr\_HIT\_RE\_500})$ $\quad \tau_{uncr} = 1512.441\ psi$

Distance to edge of project influence area: $c_{Na} := 10 \cdot d_a \cdot \sqrt{\frac{\tau_{uncr}}{1100\ psi}}$ $\quad c_{Na} = 7.329\ in$ *ACI Eqn. 17.4.5.1d*

Check if anchor bolts act in group for bond failure: *ACI 17.2.1.1*

Group_Bond_Failure_Check := if $s_1 \leq 2\ c_{Na}$
  "Bolts act in group."
else
  "Bolts act singly."

Group_Bond_Failure_Check = "Bolts act in group."

Theoretical projected influence area of a single bolt far from an edge: $A_{Nao} := \left(2\ c_{Na}\right)^2$ $A_{Nao} = 214.835\ \boldsymbol{in}^2$ *ACI Eqn. 17.4.5.1c*

Actual projected influence area for bolt(s): $A_{Na} := min\left(\left(c_{Na} + min\left(s_1, 2 \cdot c_{Na}\right) + min\left(c_{Na}, c_{a2}\right)\right) \cdot \left(c_{a1} + c_{Na}\right), n_{ab} \cdot A_{Nao}\right)$ $A_{Na} = 238.062\ \boldsymbol{in}^2$ *ACI Fig. R17.4.5.1*

Basic bond strength of adhesive anchor: $N_{ba} := \tau_{uncr} \cdot \pi \cdot d_a \cdot h_{ef}$ $N_{ba} = 14.848\ \boldsymbol{kip}$ *ACI Eqn. 17.4.5.2*

Concrete is not light weight; so, lambda-a is set to 1.0; per ACI 17.4.5.2, un-cracked bond stress may be used.

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_Na} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.4.5.3*

Factor for anchor bolts near an edge: $\Psi_{ed\_Na} := min\left(1.0, 0.7 + 0.3 \cdot \dfrac{c_{a1}}{c_{Na}}\right)$ $\Psi_{ed\_Na} = 0.918$ *ACI Eqn. 17.4.5.4b*

Factor for anchor bolts in un-cracked concrete near an edge without supplementary reinforcement: $\Psi_{cp\_Na} := min\left(1.0, \max\left(\dfrac{c_{a1}}{c_{ac}}, \dfrac{c_{Na}}{c_{ac}}\right)\right)$ $\Psi_{cp\_Na} = 0.733$ *ACI Eqn. 17.4.5.5b*

Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.

Nominal bond strength of the adhesive anchor(s): $\phi N_{ag} := \phi_{adh} \cdot \dfrac{A_{Na}}{A_{Nao}} \cdot \Psi_{ec\_Na} \cdot \Psi_{ed\_Na} \cdot \Psi_{cp\_Na} \cdot N_{ba}$ $\phi N_{ag} = 7.192\ \boldsymbol{kip}$ *ACI Eqn. 17.4.5.1b*

Check of bolt bond stress failure:

$$Bond\_Stress\_Check := \begin{array}{l} \text{if } \phi N_{ag} \ge n_{ab} \cdot T_{u\_ab} \\ \quad \| \text{“Bolt is satisfactory.”} \\ \text{else} \\ \quad \| \text{“Bolt is no good.”} \end{array}$$

$Bond_Stress_Check = $ “Bolt is satisfactory.”

Steel strength of anchor in shear (17.5.1)

Steel shear strength of anchor is confirmed above; so, no check here is necessary.

Concrete breakout strength of anchor in shear (17.5.2)

Check bolt group action for shear concrete breakout: *ACI 17.2.1.1*

$$Group\_Shear\_Breakout\_Check := \begin{array}{l} \text{if } s_1 \le 3 \cdot c_{a1} \\ \quad \| \text{“Bolts act in group.”} \\ \text{else} \\ \quad \| \text{“Bolts act singly.”} \end{array}$$

$Group_Shear_Breakout_Check = $ “Bolts act in group.”

Theoretical projected influence area of a single bolt far from an edge: $A_{Vco} := 4.5 \cdot c_{a1}{}^2$ $A_{Vco} = 127.122\ \boldsymbol{in}^2$ *ACI Eqn. 17.5.2.1c*

Actual projected influence area for bolt(s): $A_{Vc} := min\left(1.5 \cdot c_{a1} \cdot \left(1.5 \cdot c_{a1} + min\left(s_1, 3 \cdot c_{a1}\right) + min\left(1.5 \cdot c_{a1}, c_{a2}\right)\right), n_{ab} \cdot A_{Vco}\right)$ $A_{Vc} = 155.245\ \boldsymbol{in}^2$ *ACI Fig. R17.5.2.1b*

Load bearing length: $l_e := h_{ef}$ $l_e = 5\ \boldsymbol{in}$ *ACI 17.5.2.2*

Basic concrete breakout strength in shear for single anchor:

$$V_b := min\left(\left(7\left(\frac{l_e}{d_a}\right)^{0.2} \cdot \sqrt{\frac{d_a}{\boldsymbol{in}}}\right) \cdot 1.0 \cdot \sqrt{\frac{f'_c}{\boldsymbol{psi}}} \cdot \left(\frac{c_{a1}}{\boldsymbol{in}}\right)^{1.5}, 9 \cdot 1.0 \cdot \sqrt{\frac{f'_c}{\boldsymbol{psi}}} \cdot \left(\frac{c_{a1}}{\boldsymbol{in}}\right)^{1.5}\right) \cdot \boldsymbol{lbf} \qquad V_b = 6.5\ \boldsymbol{kip}$$

Concrete is not light weight; so, lambda-a is set to 1.0. *ACI Eqns. 17.5.2.2a & 17.5.2.2b*

Factor for eccentrically loaded anchor bolts: $\Psi_{ec\_V} := 1.0$ *Anchor bolts are not loaded eccentrically.* *ACI 17.5.2.5*

Factor for anchor bolts near an edge: $\Psi_{ed\_V} := min\left(1.0, 0.7 + 0.3 \cdot \dfrac{c_{a2}}{1.5 \cdot c_{a1}}\right)$ $\Psi_{ed\_V} = 0.869$ *ACI Eqns. 17.5.2.6a & 17.5.2.6b*

Factor for anchor bolts in un-cracked concrete: $\Psi_{c\_V} := 1.4$ *Wall caps are not under load, and per the wall cap design, service moment from post does not cause cracking.* *ACI 17.5.2.7*

Factor for small embedment $\Psi_{h\_V} := min\left(1.0, \sqrt{\frac{1.5 \cdot c_{a1}}{h_{ef}}}\right)$ $\Psi_{h\_V} = 1$ *ACI Eqn. 17.5.2.8*

Nominal concrete shear breakout strength: $\phi V_{cbg} := \phi_{adh} \cdot \frac{A_{Vc}}{A_{Vco}} \cdot \Psi_{ec\_V} \cdot \Psi_{ed\_V} \cdot \Psi_{c\_V} \cdot \Psi_{h\_V} \cdot V_b$ $\phi V_{cbg} = 6.28$ ***kip*** *ACI Eqn. 17.5.2.1b*

Check of concrete shear breakout failure: *Concrete_Shear_Breakout_Check* := if $\phi V_{cbg} \geq V_{post\_u}$
"Bolt is satisfactory."
else
"Bolt is no good."

Concrete_Shear_Breakout_Check = "Bolt is satisfactory."

Concrete pryout strength of anchor in shear (17.5.3)

Basic concrete pryout strength of a single anchor in shear: $\phi N_{cpg} := min\left(\phi N_{ag}, \phi N_{cbg}\right)$ $\phi N_{cpg} = 7.192$ ***kip*** *ACI 17.5.3.1*

Concrete pryout strength in shear coefficient: k_{cp} := if $h_{ef} < 2.5$ ***in***
1.0
else
2.0

$k_{cp} = 2$ *ACI 17.5.3.1*

Nominal concrete pryout strength of anchor(s) in shear: $\phi V_{cpg} := k_{cp} \cdot \phi N_{cpg}$ $\phi V_{cpg} = 14.384$ ***kip*** *ACI Eqn. 17.5.3.1b*

Check of concrete pryout strength in shear: *Concrete_Shear_Pryout_Check* := if $\phi V_{cpg} \geq V_{post\_u}$
"Bolt is satisfactory."
else
"Bolt is no good."

Concrete_Shear_Pryout_Check = "Bolt is satisfactory."